// JOURNAL OF
THE INSTITUTION OF
CIVIL ENGINEERS.

No. 7. 1936–37.
JUNE 1937.

JOURNAL OF THE INSTITUTION OF CIVIL ENGINEERS.

ADDRESS TO HIS MAJESTY THE KING.

Presented by the principal Engineering Institutions and Societies in the United Kingdom of Great Britain and Northern Ireland.

TO HIS MOST EXCELLENT MAJESTY
KING GEORGE VI.

MAY IT PLEASE YOUR MAJESTY,

We, on behalf of the members of the principal Engineering Institutions and Societies in the United Kingdom of Great Britain and Northern Ireland, desire to tender, on the occasion of your Coronation, our heartfelt congratulations and to offer our earnest wish that your Reign may be long, prosperous and peaceful.

Your Majesty has always evinced a deep interest in the Institutions that exist for the advancement of engineering science and for the better utilization of the forces of nature for the benefit of mankind. In the course of the services which you rendered as Duke of York during the lifetime of your much beloved father, His late Majesty King George V, you have been brought into direct contact with workshops, with mines and with works of engineering construction, wherein the art of the Engineer and the manual skill of the workman have combined to supply many of the needs of modern civilization.

In the assurance of your knowledge of and close interest in the work on which we are engaged, we beg leave humbly to express to Your Majesty and to Her Majesty Queen Elizabeth our loyalty and support in the onerous duties consequent upon your accession to the Throne, and we pray that Almighty God may bestow upon Your Majesties health and strength to bear these responsibilities, and happiness in the knowledge of the devotion of your people.

The Engineering Institutions and Societies who joined, by invitation from The Institution, in sending to His Majesty the King the loyal Address of congratulation on the occasion of his Coronation were :—

The Institution of Civil Engineers.
The Institution of Mechanical Engineers.
The Institution of Naval Architects.
The Institution of Gas Engineers.
The Royal Aeronautical Society.
The Iron and Steel Institute.
The Institution of Electrical Engineers.
The Institution of Municipal and County Engineers.
The Institute of Marine Engineers.
The Institution of Mining Engineers.
The Institution of Mining and Metallurgy.
The Institution of Water Engineers.
The Institution of Automobile Engineers.
The Institute of Metals.
The Institution of Structural Engineers.
The Institution of Chemical Engineers.

ORDINARY MEETING.

23 March, 1937.

Sir ALEXANDER GIBB, G.B.E., C.B., F.R.S., President, in the Chair.

On the recommendation of the Council, the members present elected by acclamation as an

Honorary Member.

Sir Robert Abbott Hadfield, Bart., D.Sc., D.Met., F.R.S., Vice-President.

The Council reported that they had recently transferred to the class of

Members.

David Barrington Brow, M.C.
Charles Hamlyn Harden, B.Sc. (Eng.) (*Lond.*).
William Hawthorne, B.E. (*Royal*).
Charles Horbury Jennings, M.Eng. (*Liverpool*).
William Cuthbert Knill, B.Sc. (*Durham*).
James Fletcher Main.
Joseph Rawlinson, M.Eng. (*Liverpool*).

And had admitted as

Students.

Dennis William Austin.
John Kenneth Ballantyne.
Frank Horace Barclay.
James Nussey Booth, B.A. (*Cantab.*).
Raymond Eric Buchan.
Thomas Hugh Young Caldwell.
Philip Sidney Carpenter.
Cyril Maxwell Clarke.
Dennis Cuttiford.
Jack Dearnley.
William Hugh Donnan.
Eric Dowson, B.Sc. (*Leeds*).
Herbert Gordon Eccles.
Peter John Newland Eve.
Cedric Leonard Flynn.
Bernard William Garrett.
Francis David Goodman.
Albert Graham.
Roderick Joffre Anthony Grealish, B.E. (*National*).
Francis Jack Haddy.
Philip Howard Hague.
Kevin Thomas Harding, B.Sc. (Eng.) (*Lond.*).
Peter Henry Hicks.
Frederick Latham Howard.
Herbert Fyfe Groom Jarvis.
Daniel Lampert.
Donald Leech.
Henry Russell Lloyd.
Richard George Medd.
Milton Rutter Moore.
Frank Norman Mottershead.
George Reginald Newton.
Iain Hamish Ogilvie, B.Sc. (*Edin.*).
James Patrick.
Edward George Porter.
Henry Banks Robertson.
William Gray Robertson.
Gordon Vernon Rose.
Robert Elliot Sewell.
Arthur Hugh Sheldon.
John Victor Garland Shilston.
Arthur Albert Smith.
Hamish Andreae Souter.
Frederick William Spencer, B.Eng. (*Liverpool*).
Francis Martin Storrar.
Hensleigh Barrett Walter.
Kenneth Herbert Whittle.
John Hutchinson Wood.

A Paper (of which a synopsis follows) on the Mechanics of the Voussoir Arch was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Authors.

Paper No. 5108.

"The Mechanics of the Voussoir Arch."

By Professor Alfred John Sutton Pippard, M.B.E., D.Sc., M. Inst. C.E., Eric Tranter, B.Sc., Stud. Inst. C.E., and Letitia Chitty, M.A.

Synopsis.[1, 2]

The Paper describes an experimental and analytical study of the behaviour of the Voussoir Arch, carried out in the Civil Engineering Department of the Imperial College at the request of the Building Research Board.

The generally-accepted basis of design for this type of structure is the assumption that no tensile stress can be taken by the mortar, and the usual method of analysis consists in designing the arch so that the line of equilibrium falls everywhere within the middle third of the depth. In the case of an arch supported either on pins or on skew-backs the problem is, upon the usual assumptions, statically indeterminate, and can only be solved by considerations of elastic behaviour. Further, the assumption that the abutments do not move may not be justified.

The first experiments described showed that, when there were no movements of the abutments, the arch behaved for small loads in a manner similar to a solid rib, whilst for larger loads it behaved as a three-pinned arch. Further experiments showed that movement of the abutments in a pin-ended arch transformed the structure into a three-pinned arch. The experiments were then repeated for the case of a voussoir arch carried on skew-backs, one end of the arch being fixed and the other end adjustable. With the span constant, the arch behaved practically as a fixed-end arch-rib ; with movement of the abutments the arch again became in effect a three-pinned arch.

[1] The Paper has already been published : Journal Inst. C.E., vol. 4 (1936–37), p. 281. (December, 1936).—Sec. Inst. C.E.

[2] Late Correspondence on this Paper can be accepted up to the 1st September, 1937, and will be published in the Institution Journal for October, 1937.—Sec. Inst. C.E.

Discussion.

The President. The PRESIDENT said that when the Council of The Institution had considered the allocation of the present evening to the discussion of the Paper on "The Mechanics of the Voussoir Arch," describing a piece of general research work, they had thought that the Members might also like to see some films of practical tests on road bridges carried out for the Ministry of Transport. They had therefore asked the Building Research Station to show those films. He desired to make it quite clear that at present the work of Professor Pippard and that of the Ministry of Transport were quite independent. Neither section had gone far enough for inferences to be drawn from one to the other.

Professor Pippard. Professor PIPPARD, in introducing the Paper, said that during the past 3 or 4 weeks the Authors had found that the opening paragraph of the Paper was somewhat unfair to certain early writers. At the end of the 18th century Boistard, a French engineer, had made experiments and had observed that arches failed by hinging about a number of sections. Those observations had enabled Lamé and Clapeyron to propound a theory of instability and to give a trial-and-error method of determining whether a given arch was suitable under given load-systems. Those investigators had only considered the case of symmetrical load-systems. Investigations had been made rather later in Great Britain by, amongst others, Messrs. Moseley, Barlow, and Snell. Snell in 1846 had read a Paper [1] to The Institution in which he had given examples of Lamé's and Clapeyron's method, and had attempted to extend it to the case of unsymmetrical loading. The result had not been too successful, for several reasons which Professor Pippard had not time to discuss that night, but from that time that aspect of the problem appeared to have been completely lost sight of in a mass of literature dealing with design methods based on the hypothesis that no tension might develop at the joints between the voussoirs. The Authors of the Paper under discussion proposed to outline the development of the theory of the voussoir arch in a later Paper.

Professor Pippard then showed a cinematograph film and a series of lantern-slides illustrating the work described in the Paper. The theoretical influence line of the horizontal thrust for an encastré

[1] George Snell, "On the Stability of Arches, with practical methods for determining, according to the pressures to which they will be subjected, the best form of section, or variable depth of voussoir, for any given intrados or extrados." Minutes of Proceedings Inst. C.E., vol. v (1846), p. 439.

Professor Pippard.

arch (*Figs. 11*, p. 298§) had been found to be incorrect for the lowest values of H/W, and considerably better agreement between theory and experiment was actually the case, showing without doubt that when the abutments were rigid and the loads were not too large, elastic theory was applicable to the problems.

A model was also exhibited by the Authors which showed the formation and transition of the "pins" in an arch mounted on skew-backs.

Further work was in progress with the object of extending the results to the stage of practical application and design, and the Authors hoped to report on that work to The Institution at a later date. He would conclude by expressing, on behalf of his co-Authors, their great obligation to the Department of Scientific and Industrial Research and to Dr. R. E. Stradling, M. Inst. C.E., for having the film made showing the experimental work.

The President.

The PRESIDENT then called upon Dr. Norman Davey, of the Building Research Station, to show films of tests on road bridges.

Dr. Davey.

Dr. NORMAN DAVEY said that no doubt many had seen references to a series of tests which the Building Research Station were carrying out for the Ministry of Transport. He had been invited to show that evening films of some of the tests, which had been made under his supervision. The films illustrated tests on two humped-back canal bridges built in 1793. One bridge was a stone voussoir arch, elliptical in form, carrying the road over a disused canal near Derby, and the other was a brick masonry arch which carried the road over the Stratford-on-Avon canal at Yardley Wood, about 5 miles south of Birmingham.

In order to apply the load, steel beams were mounted on timber cribs erected on the road surface. Upon those steel beams a timber deck was erected, and upon that deck dead load was stacked. The load was applied to the bridge by hydraulic jacks. To measure the load there was a steel block upon which was mounted a strain-gauge. Knowing the calibration of the block, it was possible to obtain a measure of the load transmitted to the arch. The deformation of the structure under various conditions of loading was observed by measuring the deflexion of the arch-ring at various points along its span, by measuring the spread of the arch and its rotation at springing, and by measuring strains in the masonry of the arch by means of gauges mounted on its intrados. Deflectometers were generally placed at the springing point and at the eighth-span points.

Dr. Davey then showed the films.

§ Page numbers so marked refer to Journal Inst. C.E., vol. 4 (1936-37). (December, 1936.)—SEC. INST. C.E.

The President. The PRESIDENT, in moving a vote of thanks to the Authors and to the exhibitor of the films, observed that bridges which were supposed to be dangerous appeared, from the films shown, to be a good deal stronger than was generally thought to be the case.

Professor Inglis. Professor C. E. INGLIS was particularly interested in Table I (p. 286§) in which the experimental and theoretical values of H/W for a solid two-pin arch-rib were compared. The agreement achieved had been astonishingly good, the discrepancy nowhere exceeding 1 per cent.; that was all the more remarkable since, from the theory given in the Appendix (pp. 304 *et seq.*§), it would appear that in calculating H (the horizontal component of the abutment-thrust), the contribution due to shear-deformation had not been taken into account. Admittedly, shear-deformation would only affect H to a small extent; it was impossible to calculate to what extent, since the dimensions of the rib were not given, but he would have thought that, having regard to the high degree of accuracy with which the values of H/W were tabulated, the contribution made by shear-deformation might possibly have been taken into account.

The behaviour of a two-pinned voussoir arch with the span slightly increased or diminished as shown in *Figs. 5* (p. 290§) and *Figs. 8* (p. 292§) was particularly interesting. The meandering of the resulting central hinge as a non-central load was steadily increased was a most illuminating conception, and when once it had been visualized the explanation was almost obvious, but in that explanation he would differ entirely from the explanation given in the Paper. The explanation set forth on p. 293§ was based upon the conception of least strain-energy, and the Authors stated that it was clear that the arch would assume the form which, for any given load, caused it to store the least amount of strain-energy. That might be clear to the Authors, but it was not at all clear to him, and their use of the principle of least strain-energy in that instance seemed to him not merely unnecessary but also quite unconvincing. The variation of strain-energy in one particular body was not being dealt with; comparisons were being made between the strain-energies in different bodies. It was true that they were built up of the same bricks, but since the central hinge was in different positions they were really different mechanisms, and although it might be capable of proof, it certainly was not obvious that the principle of least strain-energy had the extended application claimed for it in the Paper. The phenomena under consideration had no connexion whatsoever with strain or strain-energy; that was evident from the fact that the meandering of the central hinge would take place in precisely the same way if the voussoirs were made of absolutely rigid material,

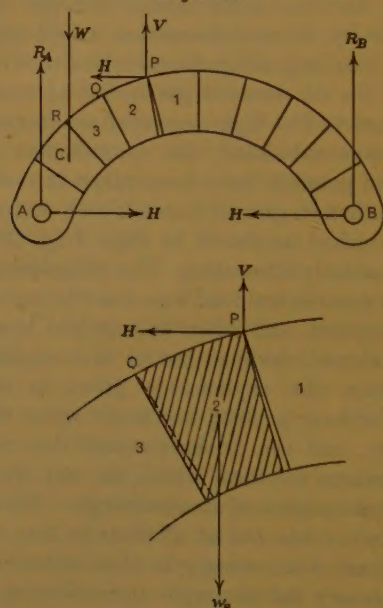
§ *Ibid.*

Professor Inglis.

in which case strain-energy could not be stored. In point of fact the problem was a simple example of rigid-body statics, and its explanation called for nothing deeper than a knowledge of how to resolve forces and to take moments.

With reference to *Figs. 15*, he would assume that when a load W acted through C, the central hinge had moved to P. By considering the equilibrium of the portion of the arch which lay to the left of P, the values of H and V, the horizontal and vertical components of the reaction at P, could at once be calculated, and, as W increased, the values of H and V would also increase. The equilibrium of

Figs. 15.



voussoir No. 2 had then to be considered: the joint between voussoirs Nos. 2 and 3 would remain closed until the sum of the moments of H and V about Q overbalanced the moment of W_2 about Q. When the overbalance occurred the joint between voussoirs Nos. 2 and 3 would open and that between Nos. 1 and 2 would close. For some greater value of W the hinge would make another move, changing its position from Q to R.

The load which caused the change in the pin-positions set forth in Table IV (p. 294§) could thus be calculated with simplicity and perfect precision. He would like to have done that, but information con-

§ *Ibid.*

cerning the weights and sizes of the voussoirs had not been available. In view of the extreme simplicity of the conditions involved he would expect the calculations thus performed to show a much closer agreement with experiment than those recorded in Table IV. Discrepancies of 10 per cent. or more were not altogether satisfactory, and strengthened the belief that the application of the principle of minimum strain-energy to those calculations, if not actually illegitimate, was certainly inappropriate. Professor Inglis.

Why had the Authors based their explanations on the principle of least strain-energy? Even if it had been applicable it was like using a steam hammer to crack a nut. The principle of least strain-energy had its uses in leading to solutions, but it gave no information which could not be deduced by ordinary statics combined with geometry, and all too often it completely obscured the true inwardness of a problem in mechanics. Such criticisms as he had made were, however, small in comparison with his admiration for the Paper.

Miss LETITIA CHITTY referred briefly to the following contribution by Professor Southwell, entitled "A Statical Theory of the Voussoir Arch": Miss Letitia Chitty.

*** Professor R. V. SOUTHWELL observed that, in place of inexact and unsatisfactory rules, based on formulas which were in fact restricted to continuous ribs of steel or cast iron, the Paper showed that an exact and reliable theory could be based on precisely that feature which at first sight would seem to complicate the problem; namely, that the mortar between the individual voussoirs could not safely be assumed to offer resistance to tension. Professor Southwell.

The one possible criticism of the Paper would be that those not previously acquainted with the problem might not be led to realize how great was the change in outlook which its results suggested. For that reason an alternative presentation of the theory might perhaps be useful, although in the nature of the case it would lead to exactly the same conclusions; that was bound to be the case, since it was concerned with the same phenomena, and merely considered them from a different angle. What followed was an attempt to emphasize that the problem was really statical, by showing that no essential feature would be lost if the voussoirs were completely rigid.

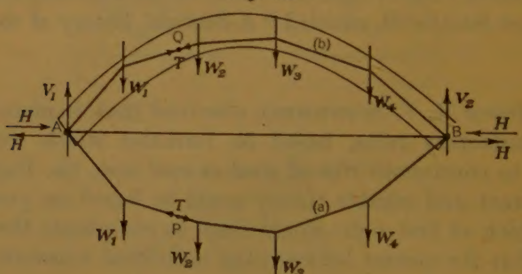
When an arch having frictionless pins at the abutments was in equilibrium, the vertical loads which it carried (including its own weight) had to be balanced by thrusts acting through the centres of the pins. Assuming that equilibrium was possible, a fundamental

*** This communication on the Paper was received prior to the Council's decision that the Paper should be read and discussed at an Ordinary Meeting.—SEC. INST. C.E.

Professor Southwell.

theorem in statics asserted that the "line of thrust" (giving the directions of the reactions between contiguous voussoirs) could be determined provided that the horizontal component (H) of those reactions were known. It was an "inverted funicular"—that was to say, it could be found by "reflecting" with respect to a horizontal plane the curve which a weightless string would assume under the same system of vertical loads, if its ends (at the same level) were held apart by horizontal forces H. *Fig. 16* illustrated that statement. Curve (a) was the funicular for the system of vertical loads W_1, W_2, W_3, . . . etc., to which the arch was subjected; the ends of the string were held fixed at A and B by vertical forces V_1, V_2 and by horizontal forces H. Curve (b) was the image of (a) with respect to a horizontal plane through A and B. Clearly, if the string (a) rested in equilibrium with a tension given by T at the point P, then a rod bent into the form of (b), and loaded by the same vertical forces and by

Fig. 16.

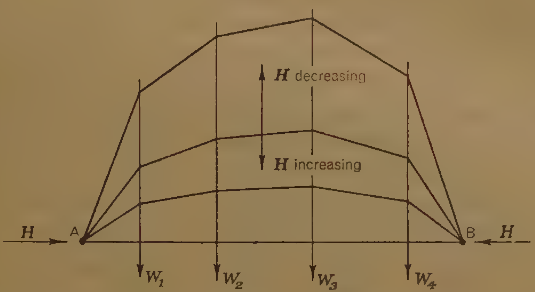


horizontal thrusts of magnitude H, could sustain those forces by thrusts alone, the thrust in the rod being T at a point Q which was the image of P. The only difference was that, whilst the form of (a) was stable for small disturbances, the form of (b) was unstable. Thinking of (b) in that way, it was not difficult to see that its form was unique for a given value of H: there could not be two configurations of equilibrium.

The theorem also permitted the effect of an alteration in the value of H to be stated: for any thrust λH the line of thrust could be obtained from (b) by a simple change of ordinates in the ratio $1 : \lambda$. Thus, if $\lambda > 1$ (H increased) the line would become flatter, whilst if $\lambda < 1$ (H decreased) it would become more sharply curved; in either event its shape would exhibit the same features (*Fig. 17*). A family of "inverted funicular" curves thus corresponded with any given system of vertical loads. Each was a possible form for the line of thrust, provided that H had the appropriate value. The flatter the curve, the larger was the associated value of H.

It was next necessary to examine whether equilibrium was in fact possible. He would show that the criterion, in respect of an arch which could not sustain tension, was whether or not a line of thrust could be drawn for the given load-system, passing through the centres of the abutment pins, and lying entirely within the arch. The meaning of the last requirement would be clear from what followed. Professor Southwell.

Fig. 17.



He would suppose first that the line of thrust went above the arch (*Fig. 18*), so as to cut CD, the plane of contact of two voussoirs A and B at a point E outside the extrados, and he would denote the thrust at E by T. That force had to be balanced by the action of B on A: if S and N were its components along and perpendicular to the face CD, then S had to be balanced by a tangential, and N by a normal, action. The tangential action, which had to be

Fig. 18.

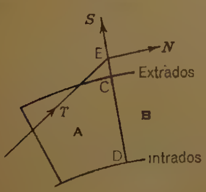
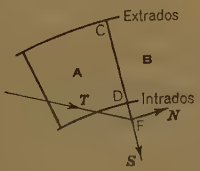


Fig. 19.



supplied by friction between the voussoirs, did not call for further consideration at the moment. If, however (as was required by the hypothesis regarding the mortar), the normal actions of B on A were wholly compressive over the face CD, then it was clear that their resultant would be a force cutting the face (at right angles) somewhere between C and D, and that force could not be in equilibrium with N. A similar argument showed that the conditions indicated in *Fig. 19*, where the thrust-line cut CD at a point F inside

Professor Southwell.

the intrados, were also inconsistent with equilibrium. Thus the following theorem had been proved: tensile strength was demanded of the mortar, unless the line of thrust lay between C and D at every face CD.

It followed from the theorem that H, when the mortar was incapable of resisting tension, was bound to have a value lying between somewhat narrow limits. Some thrust had to be applied to the arch in order to maintain it in equilibrium. If that thrust were small, the line of thrust would cross some face CD at a point outside the extrados; the adjacent voussoirs would tend in consequence to separate by rotation about a line of contact, or "hinge," at C (on the extrados); the ends would tend to spread and, unless that spread were resisted, the arch would collapse flat. If the thrust were great, the line of thrust would cross CD at a point inside the intrados, "hinging" would occur about a line of contact at D (on the intrados), and the ends in consequence would tend to approach: in so doing (if the abutments were fixed) they would cause a reduction in the value of H. It was not essential to equilibrium that the abutment hinges should be exactly at their designed positions, provided that in their actual positions they were capable of sustaining whatever thrust the arch might impose: equilibrium would result when H had attained a value such that every face CD was cut between C and D by the line of thrust, and in those circumstances it was fairly clear that friction would be sufficient to provide the necessary tangential resistance (that was, to the component denoted by S in the last paragraph).

Usually, when the loads were such that equilibrium was possible, more than one funicular (of the family mentioned earlier) could be drawn to lie wholly inside the arch, and consequently there was a range of possible values for H, bounded at the one extreme by that thrust which would make the ends approach, and at the other by that thrust which was just insufficient to prevent the ends from separating. Seen from that standpoint, no new feature was presented by an arch which was not hinged to the abutments; the only difference was that (since the line of thrust no longer had to go through definite hinge-centres, but had merely to cut lines of finite length at the abutments) the range of H, for a given load-system, was somewhat wider.

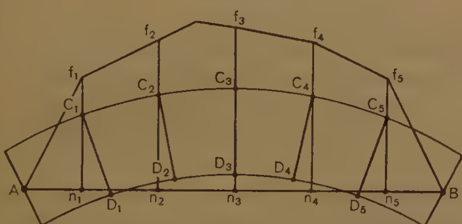
Given the vertical loads, it was easy to predict the point at which hinging would occur, and the range of possible thrusts. Taking first the case of an arch (*Fig. 20*) with pins at the abutments A and B, he would let C_1D_1, C_2D_2, . . . etc., represent the surfaces of contact of adjacent voussoirs, C_1, C_2, . . . etc., being on the extrados, and D_1, D_2, . . . etc., on the intrados of the arch. For any value H_0

Professor Southwell.

the inverted funicular corresponding with the specified loads could be drawn, and then through C_1, C_2, . . . etc., ordinates $f_1C_1n_1$, $f_2C_2n_2$, . . . etc., could be drawn as shown. He would let r_1, r_2, . . . etc., denote the ratios f_1n_1/C_1n_1, f_2n_2/C_2n_2, . . . etc. Then as H was steadily decreased from a high initial value, and in consequence (p. 10) the relevant funicular became steeper, it was clear that the extrados would first be crossed, and hinging in consequence would first occur, at that section for which r had its highest value.

In exactly the same way, drawing ordinates $F_1D_1N_1$, $F_2D_2N_2$, . . . etc. (not shown in *Fig. 20*), through D_1, D_2, . . . etc., and denoting by R_1, R_2, . . . etc., the ratios F_1N_1/D_1N_1, F_2N_2/D_2N_2, . . . etc., it could be shown that hinging would first occur, as H was steadily increased, at the intrados of that section for which R had its lowest value.

Fig. 20,



Moreover, if $r_{\text{max.}}$ denoted the highest value of r and if $R_{\text{min.}}$ denoted the lowest value of R, it was easy to show that the range of possible thrusts would be given by

$$H_0 \cdot r_{\text{max.}} < H < H_0 \cdot R_{\text{min.}} \quad . \quad . \quad . \quad . \quad . \quad (1)$$

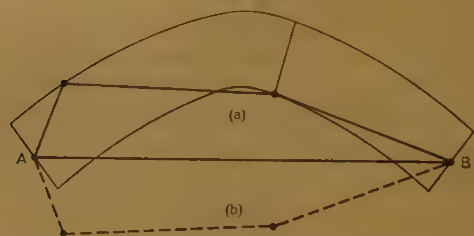
Hitherto he had dealt with cases in which equilibrium was possible, but clearly, if $R_{\text{min.}} = r_{\text{max.}}$, only one value of H was permitted by the condition (1), and no value if $R_{\text{min.}} < r_{\text{max.}}$ It was therefore necessary to consider why in those circumstances the arch was bound to collapse, even when its abutments were fixed.

From what had been shown above, when $R_{\text{min.}} = r_{\text{max.}}$ hinging could occur simultaneously at one point on the intrados and at one point on the extrados, as well as at the abutment pins; the arch would therefore behave like a linkage of three rigid members, hinged to one another and to the abutments. The statics of the problem would not be altered if those curved members were replaced by three straight rods, and the abutments (rigidly connected by the earth) by a fourth rod AB shown as (*a*) in *Fig. 21* (p. 14). The result (in effect) was the mechanism known as a four-bar chain.

Professor Southwell.

Considering with that mechanism its image with respect to *AB* (shown as (*b*) in *Fig. 21*), it was easy to see that if either one were in equilibrium, the other would also be in equilibrium under the same vertical loads. If however (as was evidently true) the equilibrium of the lower (image) mechanism were stable, so that the potential energy of the loads would be increased as a consequence of any small displacement, then the potential energy of the upper system was

Fig. 21.



bound to be decreased as a consequence of the corresponding (image) displacement, and it followed that the equilibrium of the upper system was unstable. Therefore, if hinging could occur at the intrados and extrados simultaneously, the arch would be on the point of collapse.[1]

When the arch was not pinned to the abutments, but bore on them over finite surfaces C_AD_A, C_BD_B, as indicated in *Fig. 22*, the line of thrust was no longer required to go through fixed points (A and B,

Fig. 22.

Fig. 20), but had merely to pass between C_A and D_A and between C_B and D_B. Therefore, in place of a single family of funiculars, a wider range of funiculars was now possible, limited by the occurrence of hinging at either abutment or at both. It would still be possible to deduce an inequality of the form (1), defining the range of possible thrusts, but evidently hinging at the extrados of the arch was bound

[1] It could sometimes appear that hinging became possible, simultaneously, at two points on the intrados or at two points on the extrados. Such circumstances would not, however, result in collapse, because it was kinematically impossible (when the abutment pins were fixed) that hinging should in fact occur at both points simultaneously.

to be accompanied by hinging at the intrados of one abutment or of both, and vice versa, and so four families of "limiting funiculars" called for consideration : Professor Southwell.

(i) a family passing through C_A, C_B (*Fig. 22*),
(ii) „ „ D_A, D_B,
(iii) „ „ C_A, D_B,
(iv) „ „ D_A, C_B.

Usually $r_{max.}$ would be obtained from a funicular of class (ii), and $R_{min.}$ from a funicular of class (i) ; $r_{max.}$ would be lowered and $R_{min.}$ raised as a consequence of the wider freedom of choice, so that the range of H would be wider when pins were not fitted at the abutments.

Reverting to pp. 12 and 13, a slight modification of the argument there given would be convenient when it was desired to investigate the behaviour of the arch under a concentrated load W which was steadily increased, as described on pp. 293–5§.

For a horizontal thrust H_0, he would denote by D_d the full-scale depth of the funicular drawn for the constant or "dead" loading, and by D_w the full-scale depth of the funicular drawn for a unit concentrated load replacing W. Then the depth for a concentrated load W would be $W.D_w$, and (since loads could be superposed as regards their effects on the bending moment and therefore on the funicular depth) the depth D_0 of the funicular drawn for W acting in combination with the dead loading would be given by

$$D_0 = D_d + W.D_w \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

That was for a thrust H_0: according to the argument previously set down, the depth of the funicular corresponding with any other value H would be given by

$$D = D_0 H_0 / H \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

Further, hinging would occur at a point on the extrados if $D > D_h$, where D_h denoted the full-scale height of the potential hinge above AB, the line through the abutment-pins. Similarly, hinging would occur about a point on the intrados if $D < D_h$.

Fixing attention on any one of the potential hinges, and making use of (2) and (3), it could be seen that hinging would first occur at that point when $D_h = D$, so that

$$HD_h = HD = H_0(D_d + W.D_w) \quad .. \quad . \quad . \quad (4)$$

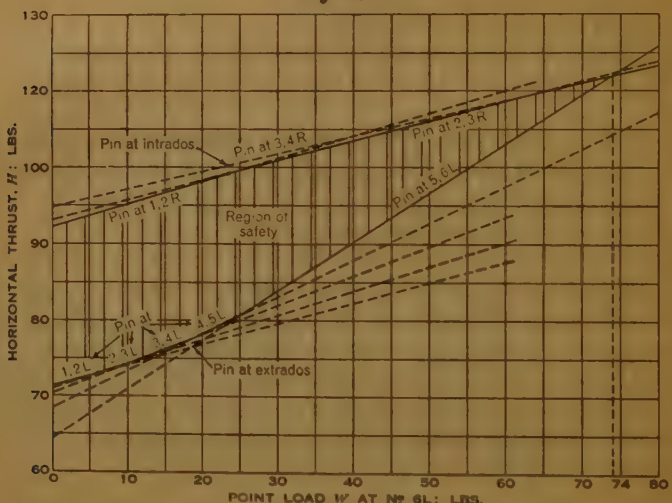
That was a linear relation between H and W, in which the coefficients H_0, D_h, D_d, D_w could be determined by standard methods. It gave a "line of safety" for the hinge-point considered, and safe

§ *Ibid.*

Professor Southwell.

values of the thrust would lie above the line if the hinge were on the extrados, and below the line if the hinge were on the intrados. By considering all possible hinge-positions, a number of lines could be determined, bounding a "region of safety" as shown in *Fig. 23*. That diagram was fundamentally the same as *Fig. 10* (p. 296§), but it had been obtained by a different method: it had been drawn for Professor Southwell by Mr. K. N. E. Bradfield, using equation (4)

Fig. 23.



to determine the positions of the different lines. Mr. Bradfield's calculations had given the following figures for a point load at 6 L, and with $H_0 = 1$:

Pin at Extrados.

Hinge-position:	1, 2 L.	2, 3 L.	3, 4 L.	4, 5 L.	5, 6 L.
D_h	1·120	1·080	1·000	0·883	0·728
D_d	80·034	76·855	70·325	60·353	46·992
D_w	0·298	0·342	0·385	0·426	0·466

Pin at Intrados.

Hinge-position:	1, 2 R.	2, 3 R.	3, 4 R.
D_h	0·869	0·833	0·762
D_d	80·104	77·499	72·227
D_w	0·256	0·215	0·176

§ *Ibid.*

As W increased, the boundary of the "region of safety" (*Fig. 23*) changed from one line to another. The different lines cut obliquely, and hence the limiting values of W could not be deduced with great precision. That geometrical difficulty was reflected in the analytical process, whereby a critical value of W was determined by eliminating H from two equations of type (4): W appeared as a quotient, and both the numerator and the denominator were small differences of relatively large quantities. However, the analytical method ought to be preferable on the score of accuracy; it led to the following results, which were directly comparable with Table IV (p. 294§): Professor Southwell.

TABLE VII.—LOADS CAUSING CHANGE OF PIN-POSITION.

Condition of span.	Position of pin changing from	Load at 6 L.	
		Experiment.	Statics.
Spread	Extrados		
	1, 2 L–2, 3 L	5·0	5·9
	2, 3 L–3, 4 L	11·5	12·3
	3, 4 L–4, 5 L	18·5	20·3
	4, 5 L–5, 6 L	24·5	24·1
Contracted	Intrados		
	1, 2 R–2, 3 R	25·0	23·5
	2, 3 R–3, 4 R	63·0	64·6

In *Fig. 23* the region of safety disappeared (that was to say, its depth became zero) when $W = 74$. That was in exact agreement with the Authors' results (p. 297§).

Throughout the preceding remarks the voussoirs had been treated as rigid, and the fact that that assumption had been found adequate to explain the experimental observations was a proof that the problem was really one of statics. Since that conclusion appeared to contradict the more usual theory (based on the notion of an elastic arch), it might be useful to consider how far the two theories were really in conflict.

The new theory suggested that the "rule of the middle third" had no relevance to the problem, in that the voussoirs would remain in contact until the thrust-line passed outside any possible hinge; that was to say, outside the arch itself. In case that conclusion should be thought to clash with the accepted engineers' theory of flexure, it was perhaps worth while to remark that what that theory really asserted was that, given continuity of the material, a tensile stress would be created if the line of thrust passed outside the middle third. Let it be supposed, however, that that tension could not be sustained, so that a gap resulted; there was no reason why the stress should

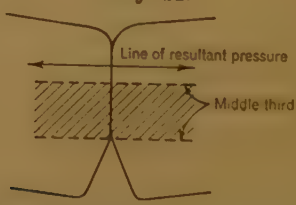
§ *Ibid.*

Professor Southwell.

not be wholly compressive over the bearing area which was left, and if two blocks were imagined pressed together in the manner of *Fig. 24*, it was clear that that was in fact what would happen. If the resultant pressure acted outside the middle third, tension would be required to close the gap, but in the absence of such tension there was no failure of equilibrium, and the gap would be inversely proportional to the rigidity of the block; that was, it would be zero when the block was completely rigid, as assumed in the previous consideration. What *Fig. 24* really demonstrated was the difficulty of making accurate allowance for elasticity in the voussoirs: it was clearly desirable that that should prove to be unnecessary, as it did according to the presentation given above.

As the resultant pressure moved nearer to one edge of the area of contact, the local intensity of pressure would increase and might ultimately attain a value in excess of the crushing strength of the masonry. That, however, was a question not for elastic theory but for experimental study, and the results of such study could be easily incorporated in rules for design.

Fig. 24.



Like the Authors, Professor Southwell had made no attempt to suggest the bearing of their work on the practical aspects of design. It was clear that its influence was bound to be far-reaching, and his own conjecture was that it would be in the direction of simplifying rather than of complicating existing practice. In another field, it might be a first step towards a theory of the mechanical principles of Gothic architecture.

Mr. Chettoe.

Mr. C. S. Chettoe said that the Paper showed how a voussoir arch loaded with dead load in conjunction with a single point-load, or a suitable group of point-loads, behaved initially as an elastic arch and afterwards, as the point-loads were increased, changed gradually to a three-pin condition as the line of thrust passed outside the arch-ring. Voussoir arches in practice were usually fixed and not two-pinned, but it was evident that the theory could be extended to cover the present case, and in fact the Authors had touched upon that extension.

Mr. Chettoe.

The usual method which had been adopted in the past for the design of voussoir arches was that of the line of pressure, which could be rapidly obtained. That method was still used to a large extent for checking the design of those arches. The line of pressure was supposed to pass through the middle third and, if so, the arch was considered to be satisfactory; since, however, in practice it was very difficult to be sure that the line of pressure was accurate, the method was not altogether satisfactory. The Authors had shown, however, that the elastic theory was applicable until the joints opened in the case of an unrestrained rib or vault. The main difficulty in calculation was in obtaining the value of the modulus of elasticity. In a brick arch the width of mortar-joints was often quite considerable, and the width and the value of the modulus of the joints had to be considered as well as those of the bricks. Experiments had shown in many cases that the value of the modulus of elasticity might be quite low. One test carried out for the Ministry of Transport at a bridge at Chester had given a value for the modulus of elasticity of about $0{\cdot}3 \times 10^6$ lbs. per square inch.

A considerable section of the Paper was devoted to the effect on the arch of spread of abutments. He felt far from certain that the amount of abutment-spread which would be met with in practice in the case of a reasonably designed arch with moderate loading on good foundations would be enough to affect the stresses on the arch very much, or would require to be taken into account in design. He had worked out a case of a five-ring brick arch having a span of 24 feet with a rise of 7 feet 6 inches. Assuming the value of the modulus of elasticity to be $0{\cdot}3 \times 10^6$ lbs. per square inch, and the total spread to be 0·01 inch (a value actually obtained in a recent test), the spread of the arch was equivalent to a fall in temperature of $3\frac{1}{2}°$ F. Assuming the arch to be loaded with the Ministry of Transport's equivalent-loading curve, the stress at the crown was increased only by about 5 lbs. per square inch, whilst that at the springing was increased by from 8 to 9 lbs. per square inch. The thrust-line in those circumstances was little affected by the spread which occurred. Spread was not normally considered in the design of reinforced-concrete arches, and it seemed to him probable that in many cases the amount of spread actually obtained in voussoir arches was not enough to have any very great effect on the stresses or in the way in which the arch behaved. The proportions of the arch in the Authors' experiments were not very dissimilar to those of a brick arch, but they were actually of steel, which was relatively a very rigid material, and probably the amount of spread which was given to the arch was quite appreciable. Nevertheless, the question of spread was one of very great importance because, although there might not be very

Mr. Chettoe. much spread in normal cases, yet in some instances considerable spread had occurred, due, for instance, to bad foundations, mining subsidence, etc. He had come across several cases; one was in a mining valley in South Wales, where both sides of the valley had moved in towards each other and the abutments of the bridges in that valley had moved in appreciably. Another interesting case was that of a bridge in Lanark, where for some geological reason the two sides of the valley had moved in, and the arch, which appeared to be perfectly satisfactory, had collapsed. In those cases some rotation of the abutments might also have occurred.

He did not feel that it was yet clear that voussoir arches in practice, whether of brick or of masonry, under reasonable loads necessarily ceased to behave as monolithic vaults capable of solution by the elastic theory, assuming that the value of the modulus of elasticity could be calculated. The Authors' experiments had been carried out on an ideal rib. Even if under working live loads an arch calculated on the elastic theory showed a tendency to open up at the joints, in practice that calculated effect might not occur, as the joint might in fact be capable of taking some tension.

The Authors' results had been obtained on an unrestrained rib, but in practice voussoir arches were frequently constructed with concrete or brick backing or haunching which extended part of the way up the vault. They were also restrained by spandrel-walls, on which parapets were usually constructed, whilst the voussoirs were covered by filling which had become consolidated by the passage of road or rail traffic. In the case of road bridges the strength of the road metalling and surfacing might have some effect on the arch. If the arches were narrow the effect of the spandrel-walls might be quite considerable in restraining them. Temperature-fall, rib-shortening, and spread of abutments would all tend to drop the arch, and the third virtual pin would rise with a reduced load at the crown whilst the application of partial live loading would tend to cause the unloaded portions of the arch to rise. If the net result of all those effects caused any particular point in the arch to rise, such movement would be resisted by the spandrel-walls and filling. If, on the other hand, the net effect were to cause a drop at a particular point, there might be some arch-action in the fill and spandrels. It seemed possible, therefore, that that restraint might change the point at which the arch, or some part of it, departed from behaviour as an elastic vault and commenced to become a three-pinned statically-determinate structure.

Mr. Wilson. Mr. J. S. Wilson remarked that the early arches had clearly been designed by those who had actually constructed them, or at any rate by those who had been very closely associated with their construc-

ion, but trouble had arisen when philosophers and mathematicians had got to work. They had had no experience of constructing rches, and the extraordinary conditions of about 150 years ago had leveloped. He would mention a bridge which Sir John Soane had ad to construct. Sir John had had no confidence in the arch, and n that little bridge every voussoir had been clamped to the adjoining oussoir by iron cramps, whilst in order to keep one voussoir from lipping on the other, he had put two cast-iron blocks, half embedded nto each voussoir, in every joint; in addition to that, he had bound he whole arch transversely with what he had called "cart-wheel" ron! Those who ought to have known the theory of the arch had een arguing at great length as to whether the depth of the arch ught to be proportioned according to the span or according to the adius of curvature at the crown; they had not been able to get any urther, so Sir John Soane, who had had to design the arch and to be esponsible for it, had not taken any risks. Mr. Wilson.

Mr. Wilson had examined arches of all types from time to time—rick arches, masonry arches, and Telford's type of cast-iron arches. The more arch-constructions were examined the more confidence was had in them. The odd thing about an arch was its extraordinary ower to suffer deformation without loss of strength. He would nention a few examples. There was Perronet's epoch-making lat arch at Neuilly, near Paris, which had a span of about 130 feet. When the centering had been removed the crown of the arch had allen 8 inches. That bridge had stood since 1774, and it had carried all the traffic to which it had been subjected. As soon as the Pont d'Alma across the Seine had been built, one of the abutments had gone down over 1 foot, but the engineers had simply re-arranged he road-level and had rebuilt some of the spandrel-walls, and the ridge had remained until a short time ago. Then there was Telford's ridge at Gloucester. That had gone down so much at the crown hat there was practically no curvature at the centre, but it still arried its load. He had seen beautiful old bridges with trees 4 or inches in diameter growing out between the joints; such bridges vere still in existence, and they were carrying quite successfully ll the traffic to which they were subjected.

By making experiments with the Authors' models, students would ain a confidence in the arch form of construction which few students f the last generation ever got, and from that point of view Mr. Wilson thought that the work described in the Paper was of great alue.

The Authors' arch was of steel with a very high modulus of lasticity; Mr. Chettoe had just pointed out that a brick arch robably had a modulus of elasticity of $0{\cdot}3 \times 10^6$ lbs. per square

Mr. Wilson. inch, or about 150 tons per square inch, as compared with about 13,000 tons per square inch for steel. Some experiments should be made with material more like that which was used in practice, and allowance should be made for the deformation of arches. In practice there was a centering which had to support the arch while it was being built, and as soon as building was commenced the centering deflected or compressed, so that the joints between the voussoirs were not perfect. Further, when the centering was taken out the arch was bound to deflect still further. That initial deformation upset the theoretical position of the line of thrust, which had such a great bearing on the stability of the arch. Some years ago he had designed some centering for a brick arch where there had been hardly any room underneath. He had designed the centering as very light steel ribs, and by temporarily loading and unloading parts of the centering, it had been kept in the right shape. By leaving gaps in the brickwork which were filled in just before removing the centering, the preliminary deformation of the centering was allowed for.

He desired to congratulate those who had carried out the experiments on actual bridges, but he thought it was hardly a fair test on the bridge to put the load all at the centre. He suggested that the Ministry of Transport should make an attempt to destroy an arch-bridge under actual working conditions; for instance, a 10-ton lorry should be driven backwards and forwards over the bridge and a record kept of how long it took to damage or destroy the bridge. He believed that, with the bridges tested, if a 30-ton or 40-ton lorry were used the test would go on for almost an indefinite period.

Mr. Adams. *** Mr. HADDON C. ADAMS observed that the objects of the experiments described in the Paper, and of others to follow as stated on pp. 281§ and 303§, were commendable if somewhat ambitious, in that they attempted to find a more rational basis of design than that founded on the elastic theory. There were bound to be many stages in the search, and there would also be a constant temptation to press the conclusions too far; it appeared in fact as if the temptation was occasionally a little too great for the Authors' restraint. The condition in the arch when two adjacent voussoirs were in contact only at their upper or lower edges was one which the designer could not contemplate with equanimity: the experiments were carried out mainly in a range of conditions which lay completely outside the designer's scope. Those conditions were interesting, but as far as the designer was concerned the arch as a monolithic structure had already failed; the designer's main interest was bound to centre in conditions at a much earlier stage.

*** This contribution was submitted in writing.—SEC. INST. C.E.
§ *Ibid.*

Mr. Adams.

In the section of the Paper headed " Review of Accepted Basis of Design " (p. 282§), the requirements as there set out would probably be fulfilled by very few voussoir arches which carried road traffic if present-day loading were considered, and if allowance were made also for temperature-variations. Fortunately, other factors played an important part in sustaining the structure, even if they were not recognized as valid in design. In the design of arches the old method of balancing the thrust-line inside the " middle third " was largely abandoned, and indeed very few real voussoir arches were at the present time being constructed for road bridges.

On p. 283§ it was stated that " Not only will the thrust of the arch cause slight spreading of the supports, but settlements of the foundations, or even earth-movements under the foundations, must be treated as serious possibilities which may cause alterations in the span after the arch is erected." The statement that " . . . . earth movements under the foundations must be treated as serious possibilities . . ." in arch-design was rather alarming. Most designers would decide at once that such a site was completely unsuited to the construction of a fixed-arch structure unless piles could be used to prevent the anticipated movement. The effects due to temperature-variations and rib-shortening (and, in concrete arches, of shrinkage and plastic yield) were similar to a slight movement of the abutments, but those movements were of a specific nature, confined to one direction, and moreover, they could be fairly accurately estimated and their effect allowed for. Movement of the foundation-bed (to any material extent) was another question ; how was it to be estimated and how was provision to be made for it ? The designer would say that it had to be prevented, or else that that type of design had to be abandoned.

The method of applying the load was rather open to criticism, and it would be very much better, if design were the object of attack, to load the arch in some way with filling placed above the voussoirs and distributed over the whole of the extrados.

The statement on pp. 285 and 286§ that " The voussoirs were provided with slots and pins to prevent relative rotation, but no other restraint was given," appeared to be misleading. Relative rotation between voussoirs would ordinarily be taken to mean rotation in the plane of the arch ; in fact, however, such movement was not restrained, and did take place—it was really the main feature of the whole experiment. Presumably the Authors intended to convey that restraint against buckling laterally was provided by the slots

§ *Ibid.*

Mr. Adams. and pins. Such a provision was reasonable, as in an actual vault its breadth would give the same effect.

He strongly criticized conclusion (g) on p. 303§, in which the Authors stated that in most actual arches there would probably be a spread of abutments, that the problem of the arch would then be statically determinate, and that any design method should therefore take into account the probability of the structure assuming the three-pinned form. Such a conclusion, jumping from an ideally-constructed laboratory model in steel (and that only a narrow strip), to an arch built as a broad vault of masonry or brick, with mortar joints and solid filling over the vault, was completely unjustified. It was a big step from one to the other and many careful experiments would have to be performed, sound deductions drawn, and criticisms answered before the designer of an arch could accept the suggestions advanced by the Authors. Was the present Paper intended to be the foundation of a new design method? The designer, of arches at least, would say that those foundations were suspect, and that it would be necessary to prove them or else to change the design.

The Authors. The AUTHORS, in reply, wished to express their thanks to Professor Southwell for the interest he had shown in the work and for his presentation of an alternative treatment for calculating the "pin" positions. They were in agreement with him and with Professor Inglis that when the structure had assumed a three-pinned form the phenomena observed could be explained without appeal to considerations of strain-energy. They would emphasize, however, that the actual construction used and illustrated in *Fig. 10* (p. 296§) was obtained solely from a consideration of the static equilibrium of the possible three-pinned arches.

No calculations involving strain-energy were required in the construction of that diagram except for the $H-W$ line for the two-pinned arch, which was only shown for purposes of argument and which would not be needed in an actual investigation of stability. In one sense *Fig. 9* (p. 294§) was unnecessary, but when the Authors first observed the sequence of pin-formation the energy-explanation had at once occurred to them, and they felt that it was not without interest and value. That particular diagram was not used for purposes of calculation, although it gave the same results as that shown in *Fig. 10*, which had been reproduced by a different method in *Fig. 23* (p. 16) by Professor Southwell. Professor Southwell drew attention to the small errors which might be introduced due to the obliquity of the intersecting lines, and that was a partial answer to the point which Professor Inglis raised in reference to Table

§ *Ibid.*

The Authors.

IV (p. 294§). The differences between the calculated and theoretical results shown there were partly due to that cause. It was, however, not always possible to state exactly the load that caused a transition of the pin, and some of the discrepancies were, therefore, due to experimental uncertainty; the Authors disagreed with Professor Inglis's suggestion that they might be due to an inappropriate treatment except to the extent that the points of intersection in *Fig. 10* might have been calculated instead of having been taken from the graph. In later work that had been done.

The method of calculation suggested by Professor Inglis and illustrated in *Fig. 15* (p. 8) might prove somewhat lengthy in application, and the Authors believed that their construction was quicker than either that method or the alternative given by Professor Southwell. Experience alone, however, could show whether or not that was the case, and it was probable that the choice of any particular method would depend largely upon the personal predilection of the user.

Professor Inglis appeared to have misunderstood the Authors' arguments on p. 293§, and they regretted that their statement was perhaps not as precise as it might have been. Professor Inglis appeared to think that the Authors had invoked Castigliano's theorem, but that was not so; the argument had been based on a general theorem in mechanics, namely, that of minimum potential energy,* and there was, they believed, no question as to the validity of that appeal.

The attention of both Professor Inglis and Professor Southwell had been directed to that part of the Paper which dealt with the arch when it had assumed the three-pinned form, and that was, in the opinion of the Authors, an aspect of primary importance and interest. It ought not to be forgotten, however, as other contributors had pointed out, that in many cases the structure might never assume the three-pinned form, and design-methods ought to take that into consideration. With certain limits of load, if the abutments were rigid, the problem was essentially an elastic one and the assumption of rigid voussoirs could not then lead to a solution of the problem, so that strain-energy analysis or an equivalent method became essential. The Authors had not arbitrarily neglected the shear-deformation which was referred to by Professor Inglis. The effect had been calculated but it had been found to be quite negligible in the particular case considered.

The other contributors to the discussion had tended to overlook the object of the Paper and had criticized it from the point of view

§ *Ibid.*
* H. Lamb, "Statics," pp. 112 *et seq.* Cambridge, 1928.

The Authors. of application to design. The temptation to do so was natural, but it had been a deliberate decision on the part of the Authors to exclude that aspect at the present stage, as they wished to concentrate on the fundamental mechanics of the problem. Further work was in progress, and an arch of 10 feet span was ready for test in the laboratory. That was built of concrete voussoirs, with mortar joints and would, it was hoped, meet one of Mr. Wilson's points and show how the properties of the materials used modified the results which had so far been obtained on the steel model.

There appeared to be some misconception on the part of Mr. Chettoe and also of Mr. Adams. Whilst it was convenient for demonstration purposes to assume that the arch span was contracted or spread, reference to p. 297§ would show that the same critical load had been obtained when no movement of the abutments had taken place. Incidentally, those two contributors did not seem to be in agreement as to the possibility of abutment-movements, but there was sufficient evidence that such movements might occur and might be sufficient to destroy the monolithic nature of the structure. The arch might, however, still have a considerable reserve of strength, and it was surely of importance to form an estimate of that, especially in the case of existing bridges. In that connexion the remark of the President on p. 7 was significant. Whether or not that contingency ought to be considered in any particular arch was the responsibility of the engineer concerned, but since there was at least a possibility of its occurrence it could not be ignored without substantial reason. Mr. Adams had placed the correct interpretation on the statement on pp. 285§ and 286§. The pins and slots were used to prevent rotation out of the plane of the arch.

In a later Paper which was in process of preparation the Authors hoped to discuss the implications of the present work in relation to design, to explain in more detail the behaviour of the fixed-end arch, and to report on experiments in which the material used was a more normal one for that type of structure. A number of the points raised by Messrs. Wilson, Chettoe and Adams had important bearings upon that next stage. The Authors thanked them for their suggestions and hoped to be able to answer some, at least, of their questions when the experiments in hand were complete.

§ *Ibid.*

ORDINARY MEETING.

6 April, 1937.

SYDNEY BRYAN DONKIN, Vice-President, in the Chair.

The Scrutineers reported that the following had been duly elected as

Associate Members.

Reginald Bradley, Stud. Inst. C.E.
Richard William Bullmore, Stud. Inst. C.E.
Henry Brewis Byers, B.Sc. (*Durham*), Stud. Inst. C.E.
James Henry Archbold Crockett, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
William Dudgeon, B.Sc. (*Edin.*), Stud. Inst. C.E.
Edward George Goldring, Stud. Inst. C.E.
James Alfred Beaumont Holborn, Stud. Inst. C.E.
Dan Campbell Holmes, B.Sc. (Eng.) (*Lond.*).
George Richard Hope, Stud. Inst. C.E.
Eric Alfred George Johnson, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Walter Sinclair Kennedy, Stud. Inst. C.E.
Gordon Hubert Lambert, Stud. Inst. C.E.
James McLachlan.
William Thomas Marshall, B.Sc. (Eng.) (*Lond.*).
John Paton, Stud. Inst. C.E.
John Leighton Sharratt, Stud. Inst. C.E.
Hugh Montgomery Thompson.
Alfred Sydney Turner, Stud. Inst. C.E.
Henry Nicholas Walsh, M.E. (*National*).
William Levinge Whately, Stud. Inst. C.E.
Harold William Willcock, Stud. Inst. C.E.
Harry Wilman, Jun., Stud. Inst. C.E.

The following Paper was submitted for discussion, and, on the motion of the Chairman, the thanks of The Institution were accorded to the Author.

Paper No. 5085.

"The Reconstruction of the Chester—Holyhead Road near Penmaenmawr, North Wales."[1]

By Cecil Lee Howard Humphreys, T.D., M. Inst. C.E.

TABLE OF CONTENTS.

Introduction.

The headlands of Penmaenbach and Penyclip are situated at the foot of the northern slopes of the Snowdon range between Conway and Bangor. The diversions described in this Paper are of the A.55, or Chester—Holyhead, road at these points. The nearest alternative (Class A) road communication between Conway and Bangor is by way of the Conway valley and Bettws-y-Coed, an extra distance of some 16 miles (*Fig. 1*).

The Penmaenbach contract was begun in October, 1930, and was opened for public use by Sir Richard Williams, D.L., the then Chairman of the Caernarvon County Council, on the 17th December, 1932. The Penyclip contract was begun in May, 1931, and was

[1] Correspondence on this Paper can be accepted until the 1st September, 1937. —Sec. Inst. C.E.

officially opened for public use by the Minister of Transport on the 5th October, 1935. The funds for both contracts were provided out of the Road Fund (85 per cent.) and by the Caernarvon County Council (15 per cent.).

The Penmaenbach Diversion.

Historical and General Notes.

Until a road was cut at the foot of Penmaenbach by Telford in the year 1826, the main route from Bangor to Conway turned inland at

Fig. 1.

Penmaenmawr and crossed the Sychnant pass. Before this date purely local communication from the eastern end of Penmaenmawr to the dunes near Conway appears to have been made over the sands around the Penmaenbach point when the tides permitted. Telford's road around the point was made by blasting away rock to form the narrow ledge upon which it is founded.

The bulk of the rock through which the old road is cut is generally well consolidated, although there are steeply-dipping cleavage-planes which form a series of extensive and thick layers of rock superimposed on one another. The actual excavation on the south side of the old road is irregular in section and is in parts overhanging. At the eastern end of the nose the layers are thinner, and, for a short way in from the surface, there are signs of the intrusion of clay into the beds. The rock is a quartz keratophyre.

The old road suffered from serious defects. The chief of these were :—

(1) Weathering and loosening of small pieces of rock from the south slopes of the road. These were liable to fall on to passing traffic.
(2) Sharp curvature and bad visibility.
(3) Exposure to very high winds, the effect of which was aggravated by the sharp corners.
(4) The breaking of the sea on to the road during storms.
(5) The formation of ice on, and icicles above, the road from the seepage water which percolated through fissures in the slopes.
(6) Danger from sheep which fell off the hillside on to the road beneath.

The disadvantages mentioned indicated that any improvement in the existing conditions would either have to take the form of protective cover or of the diversion of the road northwards, away from the headland and out to sea. A tunnel was finally decided upon, as being the best form of protection and because it would give the best alignment.

The London Midland & Scottish Railway passes under the Chester—Holyhead road at the western side of the Penmaenbach point and diverges away southwards from that point. It again passes under the road at a point nearer Conway after traversing the main part of the headland. The railway was constructed in tunnel without affecting Telford's road around the outside of the point. The present diversion of the old road inland, however, involved work in close proximity to the railway, so that blasting for the new route was liable to affect the railway tunnel, which is, for the greater part of its length, unlined. A minor fall of rock inside the railway tunnel occurred in June, 1931, after a blast.

All the disadvantages of the old road were in effect multiplied with the advent of motor traffic. The narrowness and the sharp curvature caused great inconvenience, especially during certain times of the year when the traffic was heavy, and the vibration from earlier motor vehicles tended to cause more falls of small pieces of rock from weak areas.

Early Surveys for the Diversion-Schemes.

The first survey made for a diversion-scheme was carried out directly under the instructions of the Minister of Transport, and a scheme for a tunnel was prepared on the basis of this survey. It was proposed that the tunnel should enter the west side of the headland slightly to the south of the point at which the road crosses

the railway by a bridge. Under this arrangement the railway tunnel would have been crossed by the road tunnel with, on account of the ruling gradient, an amount of cover which the railway company considered to be too small for safety. The company asked for a minimum cover of 20 feet, and this would have entailed a much longer tunnel and approaches than have in fact been built.

Another scheme was therefore developed by the Author's firm. In this scheme the existing bridge which carries the old road over the railway was to be widened and strengthened so that the new road could cross the railway in the open before the tunnel-section began. Although this entailed rather sharper curvature at the approaches it avoided the necessity for the long approaches and tunnel already mentioned, and it preserved a good alignment in the tunnel itself. Suitable portal-positions for the tunnel appeared to be available, one within 200 feet of the railway bridge at the western end and the other 565 feet further to the east. The eastern end was designed with the new road-level at the portal-position a few feet below that of the old road. An approach-ramp up to the old road-level was, however, provided so that, in the case of an accident inside the tunnel, the old road could still be used in an emergency. At the western portal the old and new road-levels were to be flush. There was some modification of this arrangement during actual construction, a part of the old road being used to provide the ramp necessary for this purpose.

A widening of the approaches on the south side at the western end westwards of the railway bridge was in the first case only looked upon as a provisional proposal, but the work was eventually authorised and was carried out, to the great improvement of the scheme. The diversion could also, with advantage, have been continued further eastwards than is in fact the case, but the extent of the work was limited by the funds at the time of construction. The plan and section of the work are shown in Figs. 2, Plate 1.

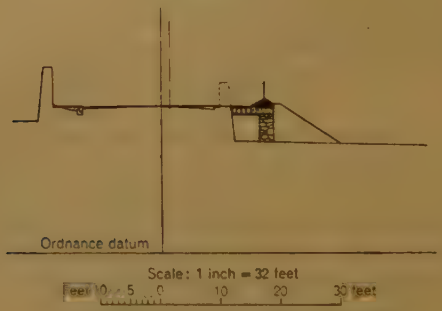
The Main Sections of the Diversion.

The work may be divided into sections as follows :—

(*a*) The western approach, up to the commencement of the work on the railway property, consists generally of the widening of the old road on the south side. The old boundary wall to the railway is maintained throughout this length to serve as the northern boundary of the new road. The length of the section is approximately 500 feet. For about a third of this length it is supported by a new retaining wall on the south side, because it has been necessary to maintain an area of garden attached to the lodge of the Pendyffryn

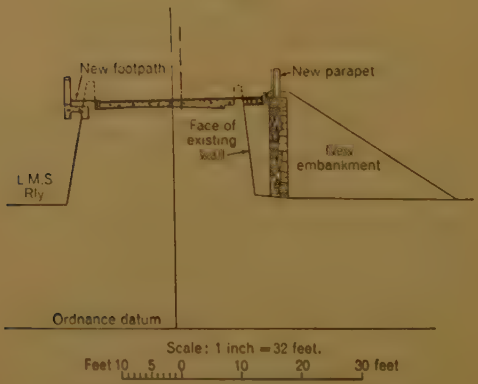
Estate. The widening of the remaining length of this section on the south side is formed on an embankment. The level of the new carriageway on this section coincides closely with that of the old road. The minimum radius of curvature on the centre-line is 400 feet. The maximum gradient is 1 in 25 rising from the lowest level of the widening, which is 22·80 O.D. This section is shown in *Fig. 3.*

Fig. 3.



CROSS-SECTION AT CHAINAGE 150.

Fig. 4.



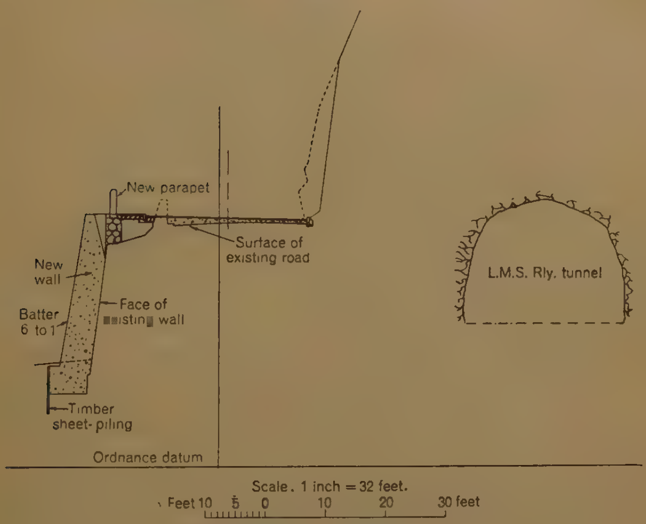
CROSS-SECTION AT CHAINAGE 500.

(*b*) The section of road up to the east side of the railway bridge; this was widened by a continuation of the embankment referred to in (*a*) on the south side, by oversailing the railway wall on the north side, and by a widening of the bridge over the railway. The work carried out on the north side also includes the strengthening of the old sea- and north retaining walls. The maximum gradient is 1 in 25 rising and the minimum radius of curvature on the centre-line is 400 feet. This section is shown in *Fig. 4.*

(*c*) The section of the road from the east side of the railway bridge up to the western tunnel-portal; this was widened partly by an embankment on the sea side and partly by excavation of the south rock-slopes at the entrance to the tunnel. The maximum gradient of the section is 1 in 250, and the minimum radius of curvature on the centre-line is 350 feet. The work carried out on this section also includes strengthening of the old sea-wall. A cross-section is shown in *Fig. 5*.

(*d*) The tunnelled section of road; this has a maximum rising gradient of 1 in 55 and a minimum radius of curvature on the centre-

Fig. 5.



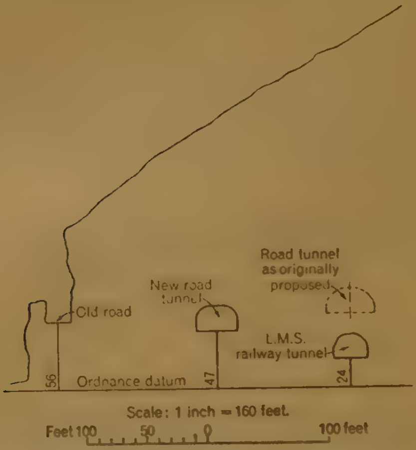
CROSS-SECTION AT CHAINAGE 710.

line of 700 feet. The summit of the diversion is virtually at the eastern end of the tunnel at a level of + 52 O.D. (*Fig. 6*, p. 34).

(*e*) From the eastern portal of the tunnel eastwards, the widening is entirely by means of excavation of the south slopes of the tunnel-approach. A sufficient width of old road was left on the north side to give the emergency one-way traffic-approach to the old road around the point, which has previously been mentioned. At the portal the new road is some feet below the level of the old road, but it rejoins the old road about 250 feet to the east. The gradient on this section is 1 in 40 falling with a minimum radius of curvature of 430 feet (*Fig. 7*, p. 34).

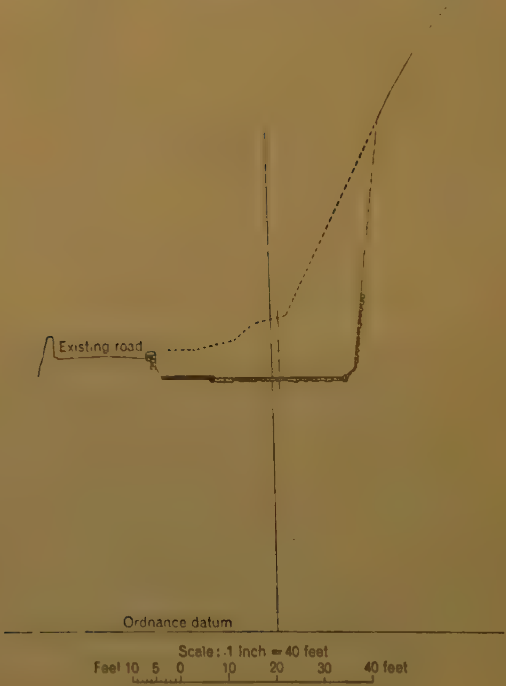
(*f*) The last and most easterly section is widened entirely on the

Fig. 6.



APPROXIMATE CROSS-SECTION AT CHAINAGE 1150.
(THROUGH GIANT'S THUMB).

Fig. 7.



CROSS-SECTION AT CHAINAGE 1480.

south side, and the new carriageway-level coincides for all practical purposes with the old. The maximum gradient is 1 in 50, and the minimum radius of curvature on the centre-line is 350 feet.

Notes on the Design.

A semi-elliptical outline was adopted for the arch of the tunnel, there being a considerable saving in excavation over a semicircular section. The headroom is 16 feet at the curbs and 21 feet at the centre-line, this being sufficient for the safe passage of the largest double-deck omnibuses. In view of the many bedding- and cleavage-planes in the rock it was decided to line the tunnel so as to prevent the loosening and consequent fall of pieces of rock due to vibration set up by traffic, which, although slight, would, it was thought, have a cumulative effect. The lining has the further object of stopping water from percolating through the rock. The design provided for the collection of water encountered during construction, so as to prevent any pressure on the lining.

In the reconstruction of the existing bridge carrying the road over the L.M.S. Railway at the western end of the tunnel, a form of concrete decking was necessary which would not involve the erection of temporary staging below. For this reason, and owing to the small construction depth available, the deck adopted consisted of pre-cast concrete segments in 18-inch widths spanning between the lower flanges of steel compound girders at 6-foot centres. The girders were connected by tie-rods at intervals and mass-concrete was filled in above the segments to the top of the girders.

The Construction of the Main Works.

The material for the embankment of the western approach was the rock excavated from the tunnel and from the approach-cutting. The new parapet-wall is built on a dry-stone core-wall about 3 feet thick, which was carried up in the embankment from the original ground-level. The filling of the embankment was not rolled, but it received some watering during construction and no appreciable sinkage has taken place, although, because of the small amount of soil or small rock in the material, voids necessarily existed to some considerable degree.

The first part of the widening of the existing railway bridge consisted of reinforced-concrete columns at centres of 15 feet and 16 feet, carrying reinforced girders. The space between the girders and the road was covered by a reinforced-concrete slab. To obtain a bearing for the inner part of this slab part of the old wall had to be reduced in height and made up in a suitable way. It is of interest

to note that, during the excavation for the two last columns, when the face of the old wall had been removed in order to build them, a slide of dry stone occurred from the wall. For the whole height of about 15 feet there was found only a very thin face of built stone in lime-mortar, and excavated stone, which had no doubt come from Telford's original excavations, had merely been dumped in behind. Heavy traffic had been running for years close up to the old parapet-wall above this point. By means of the 5-foot footpath which has been constructed on the north side vehicular traffic is now kept some 5 feet farther away from this wall than was the case before reconstruction.

The original situation at the bridge was that, at the mouth of the railway tunnel, two heavy retaining walls existed, one on each side of the line, and the space between them had been spanned by cast-iron girders which carried concrete jack-arches. Across these the north parapet of the road ran diagonally. Owing to the widening necessary for the new scheme the loading became much greater than it had originally been and the five girders which were affected, together with the jack-arches between them, had to be removed. In their place heavy steel box-girders were erected and were embedded in concrete. Concrete segments were then placed to rest between the steel box-girders on their lower flanges and, upon the platform so formed, the additional amount of concrete necessary for the deck was placed. An asphalt sealing-coat was placed over the top of the new structure to prevent infiltration of water on to the railway line.

At the western approach the cutting is practically all to the south of the centre-line. The greatest depth of cutting (measured at the centre-line) before the fall of rock which is mentioned later was 80 feet. After the fall this was increased to 85 feet. The slopes of the cuttings were designed at a batter of 8 to 1, but owing to the natural cleavage of the rock, and for other reasons connected with the excavation and blasting operations, considerably more rock was in fact excavated. On the south side, between the south curb and the bottom of the slopes, a sloping verge was designed to be formed in the rock. During excavation this width was in many cases exceeded, and the verge had therefore ultimately to be made up with a masonry facing. At the eastern end of the tunnel the south slopes were similar but the approach is in full cutting. The slopes on the north side are low and are faced with masonry. A heavy pre-cast stone coping was laid on this masonry and a substantial wrought-iron railing was fixed. Along most of the eastern approach there is no normal pavement. Both approach-cuttings are protected from loosened rocks which may fall on to the road from the hillside by a staggered line of masonry protection walls, 1 foot 6 inches wide at

the top with a batter of 8 to 1. The height of these walls at the back was designed to be 4 feet.

After the opening of the tunnel it was found necessary to close the gaps between the walls themselves, or between the walls and the steeper parts of the headland, because sheep were able to approach the fronts of the walls to find fresh grass. On many occasions the movements of these animals caused pieces of rock to fall on to the roadway, whilst the sheep themselves sometimes fell on to the roadway and were killed.

After blasting, the faces of the slopes of both approach-cuttings were in a disturbed and ragged condition and were liable to slip down on the carriageway. To prevent this a masonry facing was built to consolidate them. At the eastern approach very little overburden and debris has been allowed to remain on the top of the rock between the protection walls and the new road because, owing to the steep angle of the rock, heavy rain causes small pieces to loosen and to fall on to the carriageway. Since the removal of virtually all soil and debris from in front of the protection walls nothing of importance has fallen. At times high winds have been known to bring down small pieces of rock from the slopes above.

The main dimensions of the tunnel are: length 565 feet; width between the faces of the side walls 34 feet; height above the carriageway on the centre-line 21 feet; height of the springing-level above the carriageway at the centre-line 11 feet 7 inches. The arch is semi-elliptical in form with the semi-minor axis 9 feet 5 inches in height. The mass-concrete lining is nominally 15 inches thick and over-breakage equivalent to 15 per cent. was provided for in the contract. The actual average thickness of the tunnel-lining proved to be about double the designed thickness owing to the natural conditions of the rock and to those brought about by its excavation. Generally, when exceptionally large cubications of concrete were required to fill over-breakage, the weight of the concrete point-loads was reduced by the insertion of hollow timber boxes in the lining. The construction of the tunnel is shown in Figs. 8, Plate 1.

Four longitudinal channels were formed behind the lining for drainage purposes, with connecting down-drains every 100 feet. From the down-drains the water is carried by means of longitudinal drains situated below the dado to the outside of the tunnel, and is discharged on the foreshore. In addition to the longitudinal drains there were, at distances apart of about 100 feet, transverse drains connecting the two upper longitudinal drains across the tunnel.

The concrete of the tunnel was what was termed class "A" concrete of a nominal 1 : 2 : 4 mix. The exact proportions specified

are stated later on. The side walls were concreted in lengths of about 24 feet, and, in general, the arch-concreting was done in lengths of 12 feet. A series of 1½-inch diameter steel grouting-pipes was concreted into the lining. These pipes had screwed ends for the connexion of the grouting plant. The grouting holes were spaced at a minimum distance apart of about 5 feet from one another and over a zone of a width of 16 feet on either side of the centre-line. The grouting was specified to be continued at each pipe until grout came out of the next one to it or until no more grout could be injected. It was found after the first grouting that in many places further large quantities of grout could be injected at intermediate holes which were drilled in the lining.

The original intention had been to spray the interior face of the lining with a ¼-inch rendering of white cement and sand. Owing to board-marks which showed on the face of the concrete it was, however, decided to remove them by bush-hammering so as to give a regular appearance and finally to spray on a non-crazing white cement-mixture ("Tintocrete") followed by a transparent sealing-coat. This coating was in place for the opening ceremony, but after certain later waterproofing operations, however, it had become considerably discoloured and very irregular in appearance. The final treatment of the surface of the lining was therefore done by the application of a stippling coat ("Cullamix") which left a regular but rough surface.

A pipe-duct is formed under the footpath which is paved with reinforced-concrete slabs. This extends throughout the tunnel. Across the end of the remaining section of old road, and practically adjacent to the portal, wrought-iron gates have been erected. These gates can be opened should an emergency arise, as for example a fire inside the tunnel, and the old road can then be used.

Between the railway bridge and the western portal of the tunnel the old road is supported by a retaining wall which is built up from beach-level, and which has also to act as a sea-defence wall when heavy seas are running. This old wall was strengthened at the base by the driving of oak sheet-piles from 4 feet to 6 feet out from the wall. Part of the intervening material was excavated and the space was filled in with concrete. Rock was found towards the east end of the wall and no piles were necessary. On the foundation referred to a masonry facing was built. The rock which came out of the tunnel-excavation was used and, as the facing was brought up, concrete was filled behind this and against the face of the old retaining wall.

The normal overall width of the road outside the tunnel is 34 feet, with a carriageway width of 27 feet, a pavement width of 5 feet

and a sloping guard-verge of a width (including the curb) of 2 feet. The object of the sloping verge is to deter pedestrians from walking on it and to ensure that, when a vehicle has its wheels against the curb, the body of the vehicle will be clear of the wall bounding the road.

The carriageway is formed by a 3-inch thickness of tarmacadam, laid upon concrete where rock exists or upon hard-core where the foundation is soil or loose material. Where the old macadam was sound the new tarmacadam was laid upon it. An average thickness of 6 inches of concrete was specified to be obtained over the rock, but, owing to the natural condition of the latter and to the blasting operations there was actually more concrete than this, the average throughout being some 10 inches. To ensure that very thin patches of concrete which might have cracked up later were not left under the tarmacadam, a minimum thickness of 4 inches of concrete was required at all parts, and any rock which came inside this limit had to be removed. The curb used throughout was a sandstone grit from Bacup.

Special Points of Interest.

Rock-Fall at Western Portal.—The main dip of the rock of the headland is very steep and the general formation at the western end of the tunnel consists of a series of massive layers with cleavages and small fissures, through which water percolates in wet periods. Soon after the excavation for the western approach had been begun, and following upon a blast at a point near formation-level, a section of one of these layers, amounting to some 3,000 tons of rock, was shaken away from its bed and fell on to the existing road, where a good deal of it remained; the balance finally reached the sea-shore in front of the old sea-wall. Since other fissures appeared below the one that had failed, it was considered to be inadvisable for public traffic to pass until some reliable test had been made on the remaining layers. Traffic was therefore diverted via the Sychnant pass, which has a maximum gradient of 1 in 7½ and a somewhat loose and poor-quality surface, whilst it was arranged that the pilot-heading of the tunnel, which had up to that date been driven from the east end, should be driven concurrently from the west end. It was hoped that the rounds fired for the heading would cause any similar layers of unstable rock to be brought down so that they could be removed. Cement indicator-pats were placed on one of the bigger fissures above the south side of the old road, and the section of rock which still remained on the bedding-plane from which the fall had occurred was then removed. After the headings had met, and as no further

signs of extensive movement had shown themselves, public traffic was resumed on the existing road.

Excavation of the Tunnel.—A top heading was driven throughout prior to the bulk-excavation. The nominal size of this heading was 12 feet by 8 feet.

Materials for Construction and Tests.—The rock excavated from the tunnel was used for masonry-work and was crushed and used for the large aggregate for concrete-work. Some of the crushings were used for fine aggregate but, generally speaking, an admixture of some finer sand from quarries in the vicinity was necessary. Average compression results on the various classes of concrete were as follows :—

Class A (nominal 1 : 2 : 4) 2,600 lbs. per square inch at 7 days.
" " 4,000 " " 28 "

(Both the above results were obtained with rapid-hardening cement.)

Class C (nominal 1 : 3 : 7) 1,640 lbs. per square inch at 28 days.
(This result was obtained with ordinary cement.)

The proportions specified for these mixes were :—

	Class A.	Class C.
Cement : lbs.	568	365
Sand : cubic feet	12·2	11·6
Aggregate : cubic feet	24	27

In the tunnel-lining a rapid-hardening cement was generally used.

Percolation-specimens of class "A" concrete 2 inches thick were tested and showed no percolation under heads of water varying from 175 feet in one case to 345 feet in another case. At one stage the lining of the tunnel began to approach the area in which blasting of rock was still going on, and it was therefore considered to be desirable to test the effect of blasting on the relatively green concrete ; 6-inch cubes were accordingly made for compression-tests and for further percolation-tests. The results were as follows :—

(i) *Specimen placed within 20 feet of the blasting.*—Compression-tests at 2 days : 2,197 lbs. per square inch.

Percolation-test at age of 6 days : maximum head of water with no percolation through specimen : 345 feet.

(ii) *Specimens made at the same time but not within the range of blasting.*—Compression-tests at 2 days: 2,292 lbs. per square inch.

Percolation-test at age of 6 days: maximum head of water with no percolation through specimen: 345 feet.

In general, tests were only made at periods when some special difficulty with the concrete made them appear to be necessary. It was found, for example, that the crushed fine aggregate varied considerably in dust-content, because this depended to a great extent on the state of the wind and weather, and on the cleanliness of the screens of the crusher.

Except for the tunnel-portals and dado, all masonry was built with local stone from the excavations. This stone can readily be hammer-dressed by local workmen to the necessary dimensions. The limestone for the portals and dados of the tunnel was, however, obtained from a quarry situated between Llandudno and Colwyn Bay. The biggest stones were the voussoirs in the arch-rings, which were 3 feet by 2 feet 9 inches by 1 foot 6 inches.

Blasting Precautions.—When blasting had to be done at the western approach, or within about 400 feet of the railway bridge, the railway company was notified before every blast and the lines were then blocked by members of the permanent-way gang. No blast could be made until a disc was received from the railway ganger marked "Fire the shot," and, before he cleared the line again, the ganger inspected the railway tunnel to ensure that no damage had been done.

Traffic had also to be stopped on the main road during blasting. Owing to the heavy rock-cutting so close to the narrow existing main road, great care was necessary not only during blasting but also during the clearing and quarrying down after a blast. This was one of the operations which entailed the most risk to traffic on the existing road. It took a great deal of time to train the local men occupied in this work to realize that there is a difference between shooting down rock in a quarry and blasting rock with a public road just below. There were a few minor incidents when bad accidents nearly occurred, but fortunately there were hardly any which resulted in actual injury to the men or to the general public.

For the centering of the tunnel-lining a number of steel forms giving the designed shape of the arch were used. Each of these forms was supported on four steel-joist stanchions, and normally seven forms were mounted together as a group so that 24 feet of lining could be concreted in one position of the group. The stanchions were supported on jacks and, when the concrete was considered

to have set sufficiently, the jacks were lowered and the group was moved forward to its next position. Generally speaking, 72 hours was the shortest time which was allowed before the centres were slackened. An aerial photograph of the diversion is reproduced as *Fig. 9.*

Labour Employed.—The following figures show the greatest number of men employed at any time on the various parts of the work :—

By the contractor at the site of the work : 131 men.

By the contractor in preparing limestone in the quarry : 15 men.

By the contractor at the crusher yard : 22 men.

Throughout most of the contract the contractor was required, under the terms of the Government grant, to maintain a minimum percentage of 33⅓ per cent. labour from the " depressed areas."

THE PENYCLIP DIVERSION.

Historical and General Notes.

Penmaenmawr mountain rises from sea-level to a height of about 1,500 feet. During the last 150 years the Chester—Holyhead road has passed around it between the 100-foot and the 200-foot contours, and even earlier roads than those mentioned in this Paper have generally been within this zone. The mountain-side alternates in character between rock-outcrops, banks of scree, and sheer rocky cliffs. The rock is an enstatite diorite.

The mountain, especially at the point known as the " Clip," has always been an obstacle to transport. There are various references in old records to the difficulties and dangers which were encountered in passing around it. Towards the end of the eighteenth century a road of somewhat narrow width but of stout construction was built by Sylvester. Later some further improvements were made by Thomas Telford, F.R.S., the first President of The Institution, as part of his reconstruction of the Chester—Holyhead mail-routes in the years 1815–1830. Still later various collapses appear to have occurred to Telford's road, and it was therefore re-graded further up the mountain-side.

A few years after Telford had completed his improvement of this part of the Chester—Holyhead road, George Stephenson was constructing the railway between Chester and Holyhead in this locality. The main headland had been pierced by a tunnel and a massive sea-wall from the western end of this tunnel had been almost completely constructed when, in November 1846, a very violent north-

Fig. 9.

PENMAENBACH TUNNEL.

5085.

Fig. 10.

PENYCLIP DIVERSION.

east gale so badly damaged about 500 feet of this wall that Stephenson decided to build a viaduct because he considered that it offered fewer constructional difficulties and less obstruction to the force of any similar gales. The conditions and method of construction, both prior to the storm and after it, are described in a Paper [1] which was read before The Institution. About 25 years ago the cast-iron viaduct-girders were removed and were replaced by brick arches upon the same piers.

The foot of the mountain behind this railway viaduct consists generally of a rather weak friable shale or hill-side detritus and was not then protected, so that somewhat rapid erosion occurred. This caused the collapse of the weak surface-rock and hill-side detritus overlying it. At various times these collapses became of such magnitude that the road above was endangered and had therefore to be moved to a higher level up the mountain-side. Had the sea-wall which Stephenson nearly completed in 1846 been successfully carried along in front of this section of the headland the ground behind what is now the railway viaduct would have been protected from erosion and the history of the roads around Penyclip might have been very different. The railway viaduct offers some protection from erosion but it is nevertheless not a complete one. A concrete wall was therefore built in Viaduct bay upon the advice of Sir Benjamin Baker, K.C.B., F.R.S., Past-President Inst. C.E., in 1899, and has been effective, but by the time this work had been executed the road had been so often moved up the mountain-side that it had become impossible for modern traffic-conditions.

Recent Traffic-Difficulties on the Old Road.

With the advent of motor traffic, which increased by 44 per cent. between 1922 and 1925, the old road became more and more congested, especially at certain times of the year. Flanked as it was by two moderately steep approaches with a width between the parapet-walls of less than 15 feet in many places, and with a sharp curve and a hump in the gradient which badly affected visibility, it is no wonder that complete stoppages of traffic commonly occurred in summer. It was impossible for public vehicles such as large omnibuses to pass one another on the western approach to the summit, and it frequently happened that an omnibus would come into this section from the east when another was already coming in from the west, for neither would see the other in time and no adequate

[1] H. Swinburne, "Account of the Sea-Walls at Penmaen Mawr, on the line of the Chester and Holyhead Railway." Minutes of Proceedings Inst. C.E., vol. x. (1850–51), p. 257.

warning of their approach could be given. When the omnibuses did come within sight of one another it was frequently too late to find a place in which they could pass. The traffic would then accumulate behind each vehicle, and only after long delay and difficult and dangerous backing and manœuvring on the steep gradient would traffic become free again. At times it was even necessary to remove the mudguards of vehicles in order that they could pass. Waiting queues have extended back as far as Penmaenmawr and Llanfair-fechan respectively.

Surveys for the Present Diversion-Scheme.

A preliminary survey was made by the Minister of Transport after the War, and an estimate of the cost of a diversion-scheme was prepared. This provided for the construction of a viaduct approximately in the position of the one now built and for hillside cutting-work around the headland. In the year 1928 the Author's firm was requested to make a preliminary report on the diversion. The report was based upon the survey by the Minister of Transport, but it contained a proposal that the section of the diversion to the east of the proposed viaduct should be in tunnel throughout the main headland. This section of tunnel would have been about 1,000 feet long and would have been parallel to the railway tunnel which runs through the eastern foot of the Penmaenmawr mountain below it. It was considered that such a tunnel was desirable on the following grounds:—

(*a*) That the road would be protected for all time from falls of rock from the mountain-side;
(*b*) that high rock-slopes costly to maintain in a safe condition would be avoided;
(*c*) that an alignment with the most satisfactory curvature could be obtained by this means;
(*d*) that the road would be set farther in from the sea and would thus be less liable to subsequent movement or loss of stability from that cause;
(*e*) that an open cutting for a new road below the old road would cause the possibility of the collapse of the old road during construction of the new.

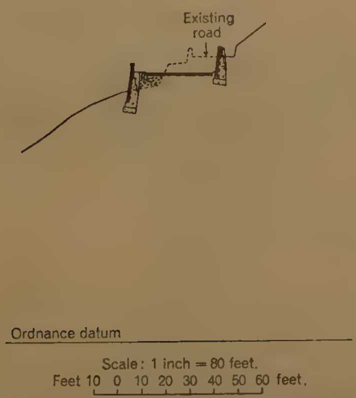
The scheme thus proposed entailed an extra cost of some £40,000 or more over the scheme for open cutting around the headland. As the Minister of Transport and the Caernarvon County Council could not see their way to provide the funds necessary for the construction of the more expensive scheme, the original open-cutting scheme was

adopted, although two lengths were tunnelled for reasons which are explained later.

A new detailed survey was therefore made for the approved scheme in the years 1929–1930. The work of surveying was difficult owing to the steep cliffs which run sheer down to the sea. In many places access to the ground to be surveyed was most troublesome and the chainmen had to be roped. The survey was made by tacheometrical methods, some 2,500 readings being taken from about thirty instrument-stations. An aerial photograph of the diversion is reproduced in *Fig. 10* (facing p. 43), and a plan and section are shown in Figs. 11, Plate 1.

The cross-sections at 1,706 and 2,300 feet (*Figs. 15*, p. 47, and *18*, p. 50) indicate the angles of the slopes below the existing main road over which a large part of the survey had to be made.

Fig. 12.



CROSS-SECTION AT CHAINAGE 550.

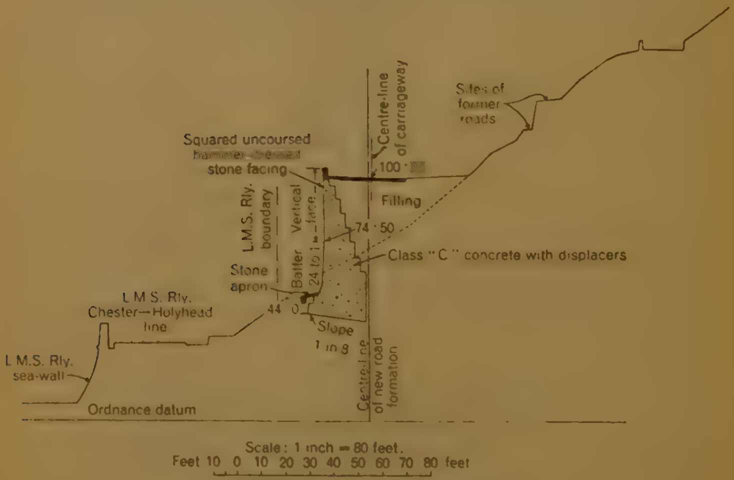
The Main Sections of the Diversion.

(*a*) The diversion commences at the eastern end of a chapel at "Garizim" which is on the eastern outskirts of Llanfairfechan. For about 450 feet the road is widened and bounded on the south by a new retaining wall. On the north side its boundaries are more or less as they originally were. The level of the new carriageway is approximately the same as the level of the old one. There is a gradient on this section of 1 in 40 and it rises to a summit at its eastern end at 109 O.D. The minimum radius of curvature on the centre-line is approximately 1,200 feet. The actual diversion of the new road from the old starts at the eastern end of this section. It is shown in *Fig. 12*.

(*b*) From the last-mentioned summit the road runs down at a

gradient of 1 in 36 as far as the viaduct; that is, for approximately another 550 feet. This section of the road is supported throughout by new retaining walls on the north side. On the south side also there is a new retaining wall which extends up to a point within 100 feet of the viaduct. This latter retaining wall is immediately adjacent to and under the old road, and part of it was actually constructed in trench across that road during the earlier operations. This section is shown in *Fig. 13*. There is a straight which begins 200 feet east of the chapel and this extends up to a point within 160 feet of the viaduct, at which point there is an 800-foot-radius curve. This curve ends just on the western abutment of the viaduct.

Fig. 13.



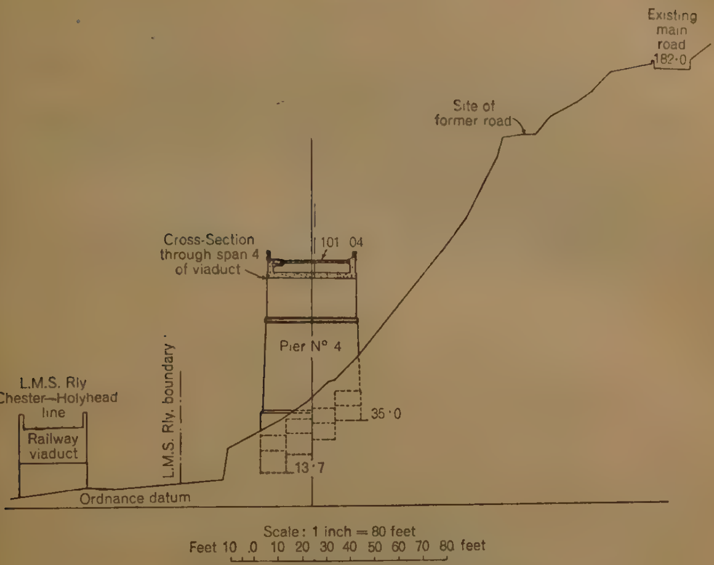
CROSS-SECTION AT CHAINAGE 980.

(*c*) The next section of road is carried on a viaduct about 700 feet long so placed that the bulk of any further falls from the unstable ground behind can pass underneath it. This length is straight and is virtually level, except for drainage gradients of 1 in 400 outwards from the centre of the viaduct towards the abutments (*Fig. 14*).

(*d*) The central section of the new road passing around the headland consists of:—

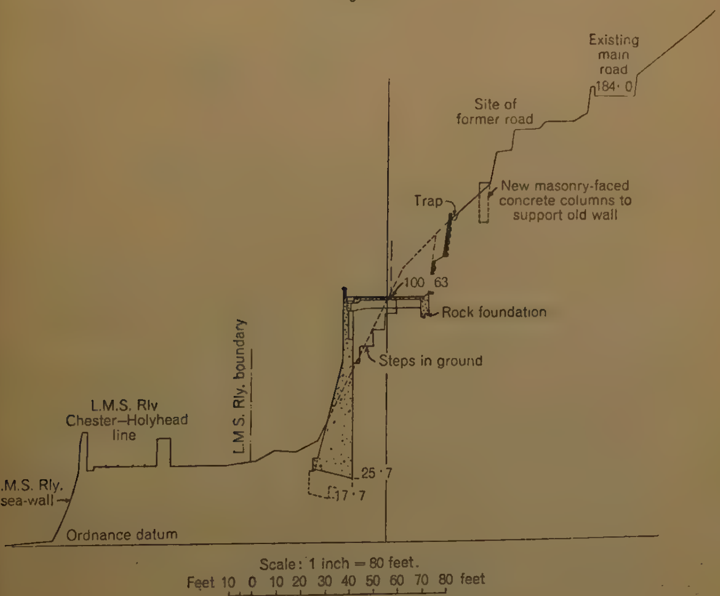
(i) A length of road carried on a reinforced-concrete deck which is supported on the north side by a vertical reinforced-concrete extension of a mass-concrete retaining wall, and on the south side by a longitudinal bearer-beam founded on the rock (*Fig. 15*).

Fig. 14.



CROSS-SECTION AT CHAINAGE 1320.

Fig. 15.

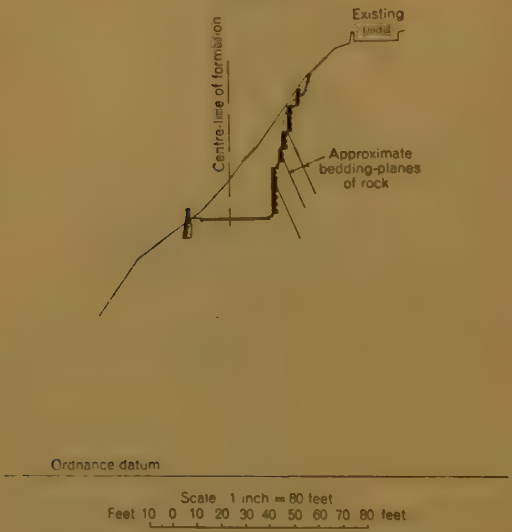


CROSS-SECTION AT CHAINAGE 1706.

(ii) A length similar to the last in so far as the north side is generally supported on a retaining wall. The south-side slopes become very much deeper than those in (i) as the tunnel is approached, and they have therefore necessitated more extensive consolidating masonry (*Fig. 16*).

(iii) A length of tunnel 113 feet long passing through a small nose of rock. This section was built in tunnel to avoid any risk that the old road might be undermined during construction, and also to avoid the necessity for the future maintenance of a very deep face of rock-cutting. Had this

Fig. 16.



Cross-Section at Chainage 1840.

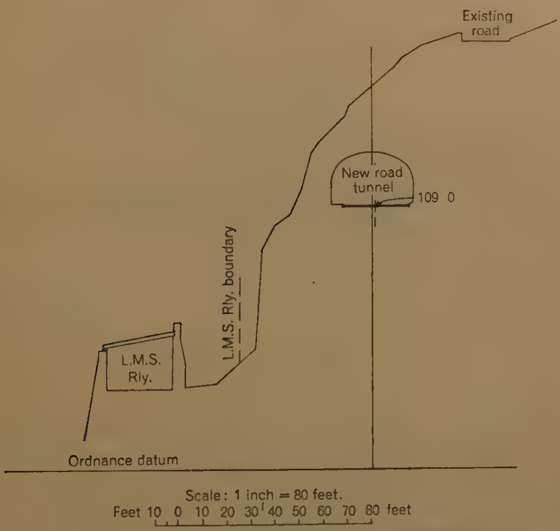
section been in open cutting it would probably not have been possible to have kept the old road open during construction (*Fig. 17*).

(iv) A section about 500 feet long formed by side-hill cutting throughout on the south side. On the north side it is supported either by a retaining wall or by foot-walls which carry a reinforced-concrete decking on masonry piers. As designed, this section was originally farther into the hill-side and nearer the existing road. When the excavation was opened out, however, it became apparent that in places the overburden lying upon the existing rock was rather deep and was liable, if left unsupported,

to slip over the rock and to undermine the existing road above. It appeared that in places the parapet-walls, and possibly the road itself, were built upon this overburden. The original centre-line of this section was therefore moved out (that is, northwards) a distance of some 11 feet. The extent of this deviation is indicated in *Figs. 18* and *19* (pp. 50 and 51).

(v) A tunnel 176 feet 6 inches in length. This tunnel was, as in the case of the previous tunnel, designed to avoid any possibility of the collapse of the existing road during construction. At this point the new road passes completely

Fig. 17.



Cross-Section at Chainage 1900.

under the old, and the method of construction made it possible to keep the old road open for traffic throughout the operations. This tunnel is shown in *Fig. 20* (p. 51).

From the end of the viaduct up to the east end of the eastern tunnel there is a rising gradient of 1 in 20, and the minimum radius of curvature is 500 feet, with a reverse curve near the east portal of the west tunnel. The total length of this division is about 1,000 feet and its level is generally some 50 feet below that of the old road. The south side of the carriageway is never in plan more than 32 feet north of the north parapet of the old road, and is sometimes as little as 7 feet away.

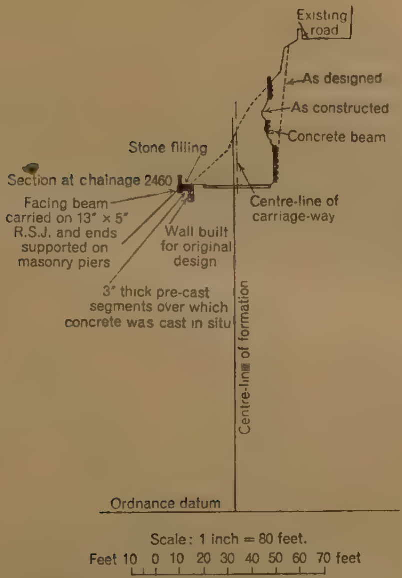
(*e*) The last section of the works stretches from the end of the eastern tunnel eastwards. Here there is some 300 feet of new road up to the point at which the new and the old roads join. On the north this stretch of road is supported partly by the rock itself, partly by a retaining wall 73 feet deep at one section, partly by a special form of column-and-panel construction on the steep hill-side, and partly by the original retaining wall upon which the old parapet

Fig. 18.

CROSS-SECTION AT CHAINAGE 2300.

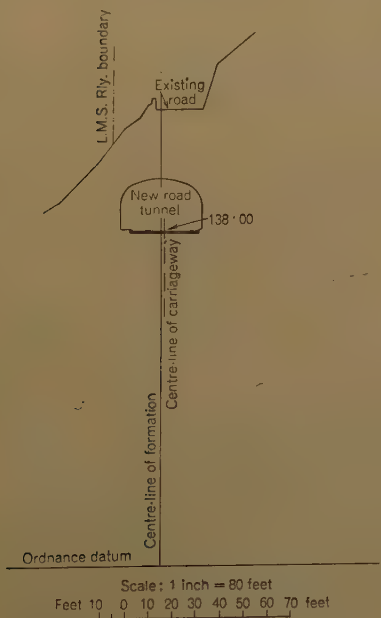
was built. On the south side the slopes are supported firstly by a section which is faced with vertical masonry backed with concrete, secondly by the under-pinning of a massive old pitched wall which was put in by Sylvester (before Telford), and thirdly by a masonry- and concrete-backed retaining wall which supports the hill-side detritus overlying the rock-slopes. This section of road has a maximum gradient of 1 in 20 and a minimum curvature of 500 feet radius. It is shown in *Fig. 21* (p. 52).

Fig. 19.



CROSS-SECTION AT CHAINAGE 2470.

Fig. 20.



CROSS-SECTION AT CHAINAGE 2560.

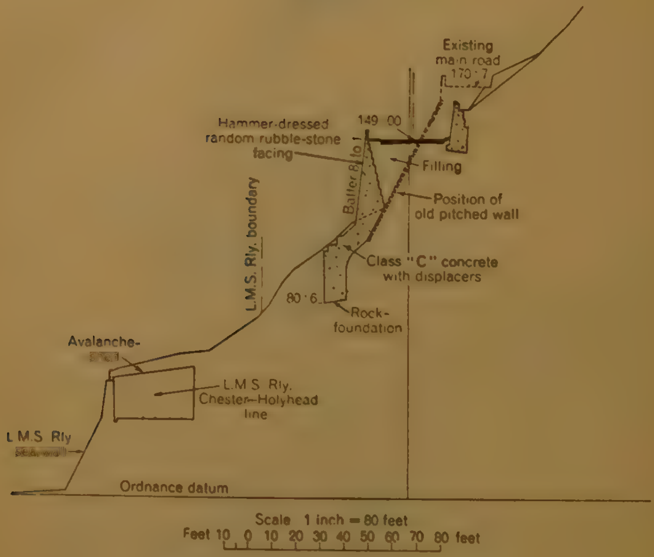
As the summit of the old road is approximately at level 193 O.D. just above the eastern tunnel, the new road reduces the total height of climb around the "Clip" by some 37 feet.

The chief dimensions of the normal construction-gauge are the same as those at Penmaenbach.

The Viaduct.—This consists of seven "fixed" arches of 80 feet clear span each, rising from piers at 92-foot centres. The rise of the arches is 20 feet and the arches are five-centred (Figs. 22, Plate 2).

In deciding the form of construction for the viaduct it was considered that the structure should be of great bulk in order to provide

Fig. 21.



CROSS-SECTION AT CHAINAGE 2780.

the necessary weight to resist shocks from large falls of debris, and that it should also be in keeping with the rugged features of the district. It was decided that the form which would harmonize best with the surroundings and would not prove unduly expensive was a series of reinforced-concrete barrel arches of semi-elliptical outline, carried on mass-concrete piers faced with local stone. A reinforced-concrete deck supported on transverse cross-walls was adopted, the spandrel-walls being of reinforced concrete faced with local stone. The intrados of each arch is made up of a five-centred circular arc approximating closely to a semi-ellipse. The arches as constructed were designed as hingeless, no movement of the founda-

tions being anticipated. An alternative design, however, was prepared embodying temporary hinges of the Mésnager type which could have been resorted to if the tests on the foundations had revealed that unequal settlement might occur. As the foundation-tests were satisfactory, the original design of rigid arches was adhered to. Provision was made for expansion and contraction of the spandrel-walls by vertical joints above the piers. The structure was designed to carry the standard Ministry of Transport loading. The stress in the arch-concrete under the worst condition was calculated to be 653 lbs. per square inch, and the maximum pressure on the pier foundations to be 6·10 tons per square foot. The design of the foundation of the western abutment presented a problem in that the trial borings had shown that hard shale was not likely to be encountered at this point above a depth of about 40 feet, the overlying debris consisting of material through which it would be practically impossible to drive any form of pile. It was decided that the most economical form of foundation would be to support the abutment on concrete "legs," formed by sinking shafts each 5 feet 6 inches square down to the hard shale and forming enlarged bases at the bottom of these by undercutting. The shafts were filled with suitably reinforced concrete, and the tops of the shafts supported a reinforced-concrete raft on which the abutment was built. The maximum pressure on the shale at the base of the pillars was calculated to be 8·7 tons per square foot.

At the eastern abutment, suitable rock-foundation being near to the surface, there were no special difficulties from the point of view of design, but to the east of this abutment, in order to avoid the necessity for an exceptionally high retaining wall, a composite form of construction was adopted comprising a mass-concrete wall of reasonable height supporting reinforced-concrete columns and panel-walls faced with masonry, which carried the roadway by means of a beam-and-slab construction spanning between the wall and the hillside. By this hollow form of construction the necessity for a very heavy retaining wall was obviated, whilst the continuity of the external appearance was maintained.

The mass-concrete retaining walls in the various parts of the scheme, where they were founded on hill-side debris, were designed for an angle of repose of 35 degrees and a maximum foundation-pressure of 3 tons per square foot. In the case of the north walls supporting the roadway on steep sidelong ground, the foundations were taken down to a minimum depth of 10 feet into the debris.

The six piers are each founded on compact shale, in some cases blue, and in others brown. The maximum height from lowest foundation-level to parapet-coping level is 96 feet 6 inches. Some

notes on tests on the foundations are given later in this Paper. The foundations each have a minimum total bearing-area of approximately 1,340 square feet. On the north side they are taken down behind the existing sea-wall to beach-level and are thence stepped up in four steps. The exact depth of the foundations depended on the material encountered, the lowest level being –8·0 O.D. at pier No. 6, from the western end. The eastern abutment is built partly on brown shale and partly on the natural rock. This foundation is also stepped, but the steps are much deeper than is the case with the piers and, on the south side, the underside of the foundation is only 41 feet from formation-level.

The western abutment (Figs. 23, Plate 2) is founded on reinforced-concrete columns 32 feet long, which have already been mentioned. They are carried through the overlying hillside-scree and detritus into the underlying blue shale. The columns are in three rows of three with the tops of each row 7 feet 6 inches higher than those of the row below. On these nine columns there is a reinforced-concrete raft which has a total area in plan of about 1,600 square feet. On the raft there is a mass-concrete abutment with a masonry facing up to the springing-level of the arches. Above that level the road is supported on a series of cross- and spandrel-walls constructed of reinforced concrete. The spandrel-walls have a local-stone masonry facing.

Pipes were left in the reinforced-concrete columns and, before the centering of the viaduct was slackened, the piers were grouted under pressure. Some 50 tons of grout were injected under the raft and around the piers at a maximum pressure of 200 lbs. per square inch, and, except in one or two places where there happened to be a free outlet, this was continued until no further quantity could be injected.

The piers and the eastern abutment are similarly built with reinforced-concrete cross- and spandrel-walls. The main reinforcement of the arches proper consists of 1-inch diameter bars spaced at 6-inch centres at the top and the bottom of the arch-barrels. The reinforced-concrete roadway-slab carrying the carriageway, the footpath, and the verge over the arches is 10 inches thick and is cambered on the upper side. The slab is supported by reinforced-concrete cross-walls and masonry-faced reinforced-concrete spandrel-walls. The cross-walls vary in thickness from 15 inches to 18 inches, and their centres are 6 feet apart. The filling over the arch up to the underside of the tarmacadam carriageway is of solid concrete for 16 feet on either side of the centre of each arch.

The masonry facing of the viaduct is in squared uncoursed masonry built of the local Penmaenmawr granite, but the quoins, the copings

and the voussoirs are of artificial stone. The artificial stone is composed of class "A" concrete, and was made on the works from crushed aggregate from the Penmaenbach excavations and aluminous cement. The exposed faces of the blocks have the outer skin of the concrete removed to show the aggregate. It was originally intended to use natural granite, but, owing to the size of the stones required, difficulties arose and the artificial stone was substituted to save both time and expense. It resembles the natural stone closely in colour.

With the exception of the solid concrete over the centre of the arch almost all the reinforced concrete above string-course level is class "R," having a nominal 1 : 1½ : 3 mix. The specified proportions of this mix were as follows:—

Cement	668 lbs.
Fine aggregate	11·3 cubic feet.
Coarse aggregate	23 cubic feet.

Some notes on tests on concrete, etc., are given later in this Paper.

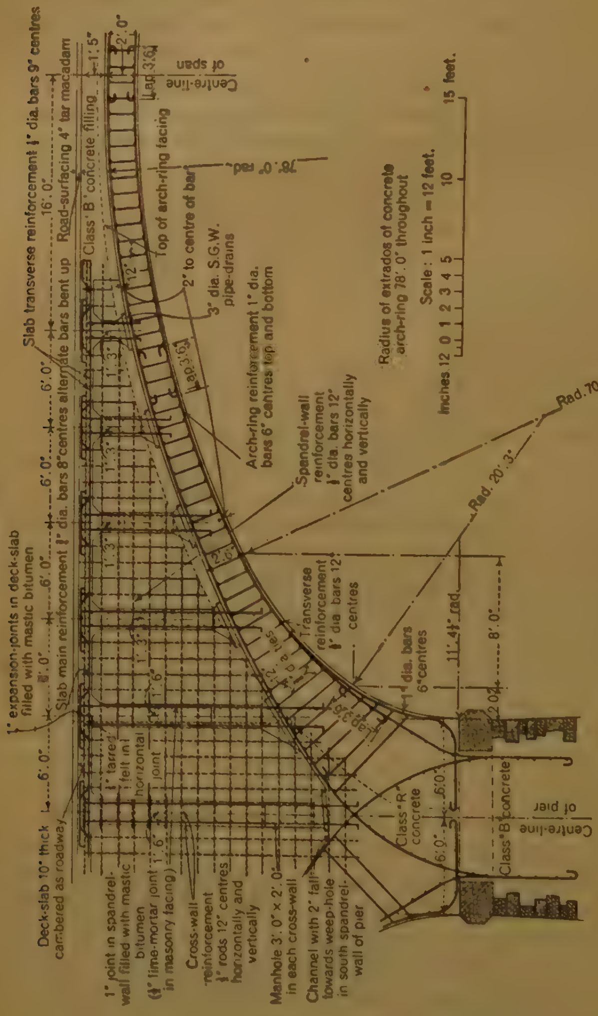
Manholes are provided in the verges to give access to the underside of the deck-slab for examination purposes, and the extrados of each arch has been treated with a bitumastic compound mixed with cement. Provision is made for drainage of condensation and other water through openings in the cross-walls down to the piers or the abutments, where outlets are provided. These outlets and the ventilation-openings in each span are filled with a bronze ventilation-grid. Four gulley-gratings are provided on the viaduct for the removal of storm-water and the drainage from these is carried through the south side of piers Nos. 1 and 5 to cast-iron down-pipes. Above the arch-ring at the end of each span expansion-joints are formed right through the upper structure; that is, through the spandrel-walls and the deck-slab. These joints are filled with a bitumastic compound. The details of the reinforcement in the arches are shown in *Fig. 24* (p. 56).

There are sixty-three voussoirs to each arch-ring. These were built on the centering and the keystones were placed before any concreting was done. A thickness of joint of ½ inch was allowed. The voussoirs were cast in steel moulds made to dimensions taken from the setting out of the arch-ring by the makers of the centerings. By this means the voussoir arch-rings closed perfectly without the necessity for adjustment of the keystone width. Two 1-inch steel bars were inserted in each of the voussoirs during casting, and these form anchorages to the main concrete.

The relative positions of the piers were carefully checked at frequent intervals by the resident engineer's staff, so that in no case was there any appreciable deviation from their true position

up to the string-course. In consequence the minimum amount of adjustment was necessary on the laggings to enable their outside dimension at springing-level to coincide with the true span.

Fig. 24.



SECTION THROUGH HALF ARCH-SPAN, SHOWING REINFORCEMENT.

Retaining Walls.—The normal types were built to a batter of 8 to 1 on the face, the thickness being increased by 6 inches in every 3 feet. The walls were constructed of class "C" concrete, faced with masonry similar to that on the viaduct. In certain sections the facing is random and not squared work.

The foundation trenches were concreted up to a height which varied with the nature of the subsoil. Above this level the concrete of each step was filled into a space between a previously-built masonry facing and a dry-stone drainage backing, so that the latter acted as shuttering to the concrete. Generally speaking, the face masonry was brought up about 3 feet at a time, the dry-stone backing was then placed, and finally the concrete was filled in.

Special Walls.—From chainage 900 to chainage 1,000 there is a special type of retaining wall with its face to a general batter of 24 to 1 only. This steep batter was necessary in order to keep this part of the structure outside the boundary of the railway company's property as there was very little room.

Another special type of retaining wall was built immediately to the east of the eastern abutment. Here the thickening of the wall from the top downwards all takes place on the outside face, which is built at a batter of 4 to 1. This particular wall was originally designed as a reinforced-concrete structure with counterforts and with a profile similar to that of the abutments. The base of such a structure would, however, have extended nearly to the centre-line of the new road, and it was subsequently decided, therefore, that by building the wall in mass-work,

(1) greater stability and increased resistance to thrust from the viaduct on to the east abutment could be obtained;

(2) the open faces of the necessary excavation on the steep hill-side would be reduced in height;

(3) the somewhat difficult excavation and construction could be done in shorter and therefore safer sections than would have been otherwise possible.

Retaining Walls between the Tunnels.—On the original alignment small short sections of retaining wall were designed to support the filling at points where the hillside fell away very steeply, and these walls were built. It was, however, subsequently decided, for reasons which have already been explained, to slue out the centre-line between the tunnels, and it then became necessary to build other walls outside the first.

In view of the good foundation which existed, and of the amount of stone suitable for building dry-stone walls which was coming from the excavation, a special form of construction was adopted for these additional walls. Steel-bar anchorages were drilled into the rock at a few feet apart and the rock was benched out so that a base upon which a masonry facing could be built was formed. The masonry facing was built plumb and the dry-stone backing was made of a sufficient thickness to hold any thrust from the filling. Concrete was then

filled into the gap which had been left between the cement-masonry facing and the dry-stone work. Selected filling without any soft material in it was used behind this form of construction.

The vertical face of the masonry made it possible to keep the toes of the walls much farther up the steep slopes of the mountain than would have been possible with a battered face; a much smaller area of masonry was necessary, and the great thickness of the dry-stone filling reduced the amount of back filling and therefore any overturning forces.

North Retaining Wall to East of Eastern Tunnel.—The original design to the east of the eastern tunnel called for a reinforced-concrete semi-viaduct. The north side of the road would have been supported on piers carrying beams built at their southern extremities into the old pitched wall constructed by Sylvester. This would have entailed the disturbance of this old wall, and, after the exposure and careful examination of the old work, it was decided that this operation could only be safely done during the period when the road was due, under the terms of the contract, to be closed completely. Means were therefore sought by which the new road could be constructed without disturbing the old wall and without stopping traffic on the old road during the construction. In the result the centre-line as originally designed was slightly modified, and it thus became possible to construct a massive retaining wall to support the north side of the new road without disturbing the latter or closing it to traffic at all.

As the main bulk of the new wall had to cover the old pitching the method adopted was :—

(*a*) to construct a thick apron-wall in short sections so as to stabilize the foot of the old pitched wall;

(*b*) upon this apron to build a thick and heavily-battered section of wall lying against and bonded to the old pitching. This wall in its turn formed a foundation upon which could be built the main wall to hold the necessary filling in front of the old pitching. This work is shown in *Fig. 21* (p. 52).

The South Retaining Wall from Chainage 2,700 to 3,000.—The original design of this wall was similar to that of the walls first described. On opening out the excavation it was, however, found that the ground would stand very much better here than had been anticipated. Advantage was therefore taken of the great designed thickness of the wall, and it was decided to reduce the height materially and to build a trap-wall which would catch any material which fretted away from the excavated slopes above. Some dry-

stone work has also been built against the slopes above the level of the wall wherever there are signs of seepage-water which might tend to disintegrate them.

Some Special Supporting Works.

Column-and-Beam Construction.—Near the western portal of the eastern tunnel and the eastern portal of the western tunnel, on account of the steep slopes and with a view to reducing the excavation in such places to the minimum, a special method of construction was carried out. A thin concrete-and-masonry stump-wall was constructed near the curb-line, or alternatively the first wall which had been built before the sluing out of the centre-line had been decided upon was used as a stump-wall. In front of this, at 20-foot centres, masonry piers were built up from the rock almost to road-formation level. Between these piers rolled-steel joists were laid and shuttering was suspended from the joists. On the outer, or sea, side the space between the shuttering and the steel joists was then filled with aluminous-cement concrete. Between the inside bottom flanges of the joists and the stump-wall, pre-cast concrete arched segments were then placed close together. Further concrete was placed on top of these segments, the backs of the joists being encased in concrete at the same time. There was thus formed a decking upon which a parapet wall could be built, and the ordinary filling and hardcore under the carriageway or footpath could be placed.

Construction on the North Side between Chainage 2,900 and 3,000.—In this area the new work coincided very closely with the line of the old parapet wall and, owing to the steep slopes below, work outside the old parapet was liable to undermine the old road. A form of construction was therefore adopted by which the excavation was carried out in small isolated units, and dry stone was generally used instead of timbering.

Reinforced-Concrete Decking to the East of the Eastern Abutment.—The foundation for the supporting wall to the deck on the north side of the new road immediately to the east of the eastern abutment is, as has been already stated, a reinforced-concrete wall. This rises from the top of a mass-concrete retaining wall and has maximum thickness of 4 feet. It has a masonry facing and is built plumb up to formation-level. On the south side of the roadway there is a longitudinal beam of reinforced concrete supported throughout on the rock. Between the north wall and this longitudinal beam reinforced-concrete cross-beams are built at 6-foot centres and these beams in turn carry a reinforced-concrete decking. The shape of the cross-beams is specially designed to allow the normal footpath

width and to form a pipe-duct on the north side. The ground has been retained under the cross-beams so far as possible in order to ensure ample support outside the longitudinal beam on the south side. From the level of the top of the batter of the north retaining wall this ground is cut back in short steps to ensure that no outward thrust occurs upon the upper section of the main north-side wall.

The main details of the transverse beams are as follows :—

Maximum depth between main reinforcement-bars : 4 feet 2 inches.

Total number of main reinforcement-bars in beams : ten bottom bars, two top bars.

Diameter of reinforcement : 1⅛ inch.

The decking is 10 inches thick with ⅝-inch diameter reinforcement.

The Two Tunnels.—The construction-gauge of the two tunnels is the same as that of the Penmaenbach tunnel. The portals are skewed to the normal at an angle of 60 degrees to the centre-line and they converge towards one another on the north side. The tunnels are lined, as in the case of the Penmaenbach tunnel, with class " A " concrete, having a nominal thickness of 15 inches, but the actual average thickness is very much greater owing to over-breakage.

In both cases the length of the tunnel as designed has been modified to suit the nature of the rock encountered. The western ends of both tunnels have thus been extended from their designed position. The eastern end of the western tunnel is virtually in the position designed, but at the eastern end of the eastern tunnel the length has been reduced. Both the tunnels are carried through the rock with very little cover on the north side, and in order to consolidate and support the rock at and near the portals thick concrete buttresses were built before the tunnels were opened out. These buttresses were constructed in masonry-faced class " C " concrete, and they are anchored with a number of steel bars which are entered and grouted into holes drilled deeply into the rock.

At the western portal of the western tunnel and at the eastern end of the eastern tunnel trial-holes showed that there were actually places where no rock-cover existed at all, and the ground was all soft material. To deal with these cases the ground was excavated down to the top of the rock, wherever that happened to be, and concrete consolidating haunches in the form of buttress-walls were built to a sufficiently high level to ensure that adequate provision was made for any thrust from the arch. In effect this work was done to ensure that there was rock-cover to the tunnel, even though the cover had to be artificially made.

Top headings were driven in both tunnels and the larger over-

breakage cavities in the roof of the pilot-headings were filled with concrete anchored to the rock by bars drilled into the rock round the cavities. Owing to the thin cover, to fissures, to the shattered nature of much of the rock, and to the overbreakage which had occurred, this concrete filling did valuable work in consolidating and strengthening the somewhat precarious roof. It also arrested the tendency of the overbreakage to spread upwards and so to cause collapses which would have affected the existing main road above the tunnels. Where a large loss of ground occurred during the excavation for the eastern portals of both tunnels, heavy reinforced-concrete beams, anchored by bars grouted into the surrounding rock, were employed to replace the natural arching effect of the lost rock and the ground above, and to stabilize the slopes under the old road.

Side-Hill Cuttings.—Although the rock is of an extremely hard nature and is generally well cemented naturally, there are nevertheless a good many clay-filled cleavages. Where the inclination of such cleavages was away from the road the excavation could be done without much risk of serious falls, but where the cleavages inclined over the road the work had to be done carefully, as otherwise removal of rock at formation-level undermined the beds above. As these beds in turn supported other layers of rock which formed the foundation of the old road upon which traffic was running during construction, it can be seen that it was necessary to treat the slopes with great care so as to avoid the collapse of the old road. Throughout this section varying thicknesses of overburden, consisting of soil and loose debris, with a binding of heather and grass, lay over the rock. When excavation was started it was found that this overburden was liable to slip and to undermine the parapet wall of the existing road. This was in many places not founded on the rock but was merely built on the overburden. Indeed, in some cases it was simply upon a brushwood filling of hollows in the rock.

At a stage, therefore, when working access had been obtained over the whole area between the two tunnels nearly down to formation-level, and following upon a heavy fall, a modified system of excavation was instituted. The rock was drilled parallel to the centre-line, and for a height up to 15 feet from formation-level, to a vertical face 17 feet from the centre-line; that is to say, to the normal formation-width. If the bulk which remained on the free side of this setting of holes were extensive, or if there were a great deal of rock left above the 15-foot height already mentioned, extra holes were fired to reduce it. The firing was done by small delayed-action charges in order to cause the least shaking effect. After each round, which generally gave an advance of from 3 to 6 feet, the resulting situation would be examined and if the rock to the

south of the 17-foot profile had moved or was materially shaken, a section of masonry, sometimes backed with concrete, would be built up in rapid-hardening cement before the next cut parallel to the centre-line was made. By this means the cutting to full formation-width was continued safely. As might be expected, the rock was usually most shaken where a natural gully in the surface of the mountain-side had formed a water-course.

This system of trimming was continued for some time, but, as the central mass of rock between the two tunnels was approached, it became more unsatisfactory. The situation then was that almost immediately below the old road there was a high and extensive face of broken and fissured rock, with weak clay-seamed layers which lay forward over the position of the new road. The top of this face was so near the old road that a relatively small disturbance of rock at the bottom might have endangered it. As the old road was also in side-hill cutting any such collapse would have resulted in its being closed for many months, the only alternative route available being the extra 16 miles via Bettws-y-Coed.

The centre of this specially weak rock was at cross-section 2,300 and the probable result of a collapse of the old road at this point can perhaps be best appreciated by an examination of that cross-section (*Fig. 18*, p. 50). In view, therefore, of these difficulties, it was decided, as has already been explained, to swing the centre-line out. This had the additional advantage that, in places where the full formation-width had nearly been obtained, the normal 2-foot guard verge would be considerably widened, and traffic would thus be kept farther away from any possible future rock-falls.

The next step was to support all the overhanging rock with buttresses of concrete faced with masonry. Wherever the excavation and blasting had loosened rock or had affected its natural solidity, masonry-faced concrete walls or buttresses were built. In some cases, in order to avoid removing rock to form a foundation, these walls were anchored to the sounder parts of the rock with steel anchorages grouted into drilled holes. The shifting of the centre-line out towards the sea also resulted in the reduction of the height of the rock-slopes which had to be faced with masonry, and it reduced the quantity of excavation. The vertical profile which was substituted for the batter of 8 to 1 gave these advantages.

At the tunnel-portals the old road was, in plan, very close to the new road, and a system of parallel holes such as might be used for the final trimming of the side excavation of a tunnel was adopted. A template of a profile similar to the tunnel-lining was used, and the holes were drilled about 18 inches apart and 18 inches deep for each round. It was found that, by this method, not only could the rock

be trimmed to a vertical face at the formation-width, but it could be cut out neatly to a tunnel-profile; that is, with the rock overhanging where it was sufficiently sound. The advantage of this was that the rock on the seaward side of the old road and almost vertically below it was not disturbed.

To the west of the western tunnel the rock on the south side of the formation was found to be much fissured with intrusions of clay and with the beds leaning forward over the new road. It was not practicable to shift the centre-line here. This unreliable area extended for a distance of about 140 feet from the portal of the tunnel, and any falls of magnitude at this point might have reached the railway line below. The line here has not even the protection of a covered avalanche-shed such as it has at other points. In order to deal with this area safely it was arranged that excavation should start at a point on the undisturbed slopes which would be given by a line carried up at 4 to 1 from the foundation level 17 feet from the centre-line. The excavation was then done in restricted lengths of section. Much of the rock required no explosives and was simply barred down.

The cutting was carried down as nearly as possible vertically. When a depth of face at which the rock appeared to require some support had been formed, this was supplied by masonry-faced concrete anchored to the more solid parts by steel bars. The cutting was then continued further downwards by another almost vertical cut, which began at a suitable distance in front of the last-built supporting wall, until this next face of excavation in its turn required some support. Wherever sound rock occurred it was left and was used as a base to which bars could be fixed as anchorages. The series of walls put in in this area is therefore not of a continuous nature or to a continuous plan or elevation. Before the last excavation at the bottom of the slopes was done, however, all the isolated sections of masonry were linked up at various levels by means of concrete with longitudinal anchorage-bars.

Nearly all the consolidation walls on the southern slopes are taken up above the top-level of the ground above them to form a trap about 3 feet in height to catch stones or other debris which may fall towards the road. It is not possible to make a trap of sufficient height and strength to safeguard the new road against every possible fall, but as matters now stand, any rock which may drop from behind the old road will generally be stopped by the parapet of the old road. If this should collapse, it, in its turn, would be caught by the traps on top of the new walls.

Road-bed, Carriageway and Footpath.—Where the formation is on rock the road-bed is of class "C" concrete. This concrete was

specified to be of an average thickness of 6 inches. Where the formation is on filling the road-bed generally consists of 12 inches of hardcore pitching. The footpaths, except where special ducts were formed on the viaduct, in the tunnels, etc., were specified to be laid on a bed of 9 inches of hardcore.

In the case of the pavements on the viaduct, in the tunnels, and in the section immediately to the east of the eastern end of the viaduct, a special duct of dimensions 3 feet by 1 foot 6 inches was formed. This duct has temporarily been filled with ballast. The footpath-slabs over the ducts are reinforced. The carriageway throughout the work is laid with a 4-inch thickness of tarmacadam.

The curb between zero and cross-section 500 is a sandstone grit from Bacup of a similar quality to that in the Penmaenbach tunnel. The curb between cross-section 500 and the eastern end of the work is of granite from Trevor, Caernarvonshire, of 12-inch by 6-inch nominal section, and the stones are generally not appreciably less than from 2 feet 6 inches to 3 feet in length. Except in the case of the special paving on the viaduct, in the tunnels, and the section at the eastern end of the eastern abutment, the footpath is paved with 2-inch pre-cast concrete slabs. It was originally specified to be laid in tarmacadam, but this was altered with a view to avoiding unsightly reinstatements after service mains had been laid. A gas-main and Post Office ducts have been laid since the opening of the diversion.

Parapet Walls and View-points.—Except at the tunnels, there is a parapet wall on the north side of the road almost throughout the diversion. It is built in squared rubble local-stone masonry except where the outside of a retaining wall is in random work, in which case the work is also random on the outside. Where the parapet is built above a battered wall the batter is continued up to coping-level; where the wall upon which it is built is vertical or where it is built upon the spandrel-walls of the viaduct it is of a regular thickness. The average height of the parapet from pavement level to the underside of the coping is about 3 feet. The actual effective height varies and depends upon the super-elevation of the carriageway, the minimum height being 2 feet 8 inches and the maximum height being 4 feet. The width under the coping varies from 1 foot to 1½ foot.

At both ends of the western tunnel the thickening buttresses which were built to consolidate the rock below formation-level have been utilized to form view-points which extend out from the pavement for some 10 feet or more. On the north side of these view-points a heavy wrought-iron railing has been built and a comprehensive view of the Great Orme, of Anglesey, and of the sea can be

obtained. A similar view-point has been built at cross-section 2,180. A striking perspective view of the viaduct can be obtained at this point. Generally speaking, the viaduct cannot be seen to advantage from the road itself. At the west end of the viaduct a small site has been laid with tarmacadam for use as a parking ground. This enables a few cars to be parked clear of the carriageway width.

Drainage.—A considerable amount of drainage had to be provided. The chief items were :—

(*a*) A spring at the eastern end of the work a few feet to the east of the eastern tunnel.

(*b*) The water from a small but steep depression a little to the west of the eastern tunnel.

(*c*) The drainage from a spring which emerges on the south side of the old road at a point at its summit approximately behind the centre of the viaduct.

(*d*) The water which collects on the old road.

(*e*) The water from the old road between the eastern tunnel and a point opposite the middle of the viaduct.

(*f*) The drainage from the old road west of a point behind the centre of the viaduct.

(*g*) The water from another spring.

(*h*) Ordinary surface-water drainage from the carriageway.

Tests on Foundations.

Some notes on foundation-tests may be of interest.

(*a*) Before the design of the viaduct was completed trial borings were made in Viaduct Bay to determine the nature of the ground through which the abutments and piers would have to be sunk and the approximate depth at which a satisfactory foundation would be encountered. Three borings were made approximately on the centre-line of the viaduct and one to the north of it (in addition, a trial-hole was sunk to the south of this line). Compact shale was found in every case at levels varying from O.D. to + 30 O.D.

(*b*) At a later date tests of the bearing capacity of the shale were made by means of a hydraulic cartridge near the site of pier No. 6, and in an adit at the foot of one of the western-abutment column-shafts.

(*c*) A test was also made between piers Nos. 2 and 3 by direct loading of the shale with 150 tons of steel rails on a concrete shaft and loading platform. The total load applied was 216 tons. These tests all showed that the

shale could withstand pressures up to 12 tons per square foot with a settlement of not more than 0·35 inch. The actual sinkage of the piers and abutments after construction was negligible.

Tests on Concrete.

Each arch of the viaduct was divided up into eleven sections of a volume suitable for concreting, and the order in which the sections were concreted was arranged so as to eliminate distortion of the centering.

Two 6-inch compression-cubes were made from the concrete when it was being placed on each section of arch. One of each pair of cubes was crushed at the age of 7 days and the other was kept for a long-period test. All the cubes were crushed before the centering was struck. A summary of the results of these tests is given in Tables I and II. In each case the figures given show the average of a number of results of tests made during the period. The figures for "combined dust-content" show the percentage of dust passing the No. 100 B.S.S. sieve contained in the mixtures of sand being used. The concrete was class "R" (nominal $1:1\frac{1}{2}:3$); the actual proportions being as set out on p. 55.

Table II shows the average increase in strength of the second cube of a pair over the corresponding cube that had been tested at 7 days.

Table I.—Summary of Compression-Tests. Cubes Crushed at 7 Days.

Cubes made between dates.	Average crushing strength: lbs. per square inch.	Average density: lbs. per cubic foot.	Average combined dust-content: per cent.	Average of material passing $\frac{1}{16}$-inch and retained on $\frac{1}{100}$-inch sieve: per cent.	Average retained on $\frac{1}{8}$-inch mesh: per cent.	Water/cement ratio: per cent.
28.3.34 . 28.5.34 .	2,944	151·2		No accurate details.		
4.6.34 . 22.6.34 .	2,281	148·7	7·0	No accurate details.	55·7	52·0
25.6.34 . 11.7.34 .	3,095	149·0	5·6	27·4	53·4	51·5
13.7.34 . 31.7.34 .	3,015	147·8	6·1	37·7	60·9	47·8
10.8.34 . 28.8.34 .	3,560	148·5	7·0	40·1	55·1	47·4
29.8.34 . 2.10.34 .	3,592	147·2	6·3	31·9	59·1	49·2
Maximum	4,297	152·0	9·0	48·5	69·2	55·2
Minimum	1,919	145·5	4·3	26·5	41·1	44·3

TABLE II.—LONG-PERIOD TESTS.

Age at date of test: days.	Ratio of $\frac{\text{crushing strength at age given}}{\text{crushing strength at age of 7 days}}$.		
	Average.	Maximum.	Minimum.
90	1·74	2·04	1·68
120	1·91	2·33	1·66
150	2·07	2·12	1·98
180	2·32	2·64	2·16
210	2·63	3·18	2·23

VIADUCT BAY.

At a late stage it was decided that, as an addition to the original works, it would be desirable to strengthen the old sea-wall, which had been built under the advice of Sir Benjamin Baker, in Viaduct Bay.

This work was undertaken by direct labour, as had been the first portion of the Penyclip diversion. It consisted of a mass-concrete apron sunk to a depth of 7 feet in front of the old sea-wall and bonded to it by reinforcing bars. The concrete apron is capped by a masonry pitching built in 2 : 1 rapid-hardening cement mortar.

CONCLUSION.

The Author wishes to record the thanks of his firm to the engineering staff for the excellent way in which they carried out their difficult duties. The Resident Engineer was Mr. A. B. Taylor, M. Inst. C.E., the Assistant Resident Engineer at Penmaenbach was Mr. J. F. Carne, Assoc. M. Inst. C.E., and the Assistant Resident Engineer at Penyclip was Mr. H. H. Dixon, M.A., Assoc. M. Inst. C.E. The Author is especially indebted to Mr. Taylor for help in the preparation of this Paper. The District Engineer of the London Midland & Scottish Railway Company during nearly the whole of the period of the execution of the work was Mr. James Briggs, M. Inst. C.E., and the Author wishes to mention his ready co-operation and help. Finally, the Author would wish to place on record his indebtedness to Sir Henry Maybury, G.B.E., K.C.M.G., C.B., Past-President Inst. C.E., to the officers of the Minister of Transport and to the County Surveyor of Caernarvon, Mr. Thomas Owen, for the help that they gave.

The Contractor for both works was Mr. M. A. Boswell, of Wolverhampton.

The Paper is accompanied by nine sheets of drawings and by four photographs, from some of which Plates 1 and 2, the Figures in the text, and the half-tone page-plate have been prepared, and by the following Appendix.

APPENDIX.

COSTS OF WORK.

	Penmaenbach.	Penyclip.
(*a*) Contract price	£52,816.	£183,453.
(*b*) Actual cost to compare with (*a*)	£58,926.	Not yet ascertained.

Detail contract prices for main items were :—

	Penyclip.	Penmaenbach.
Rock-excavation in tunnels .	35/– per cubic yard.	22/6 per cubic yard.
Rock-excavation in approaches.	15/– ,, ,, ,,	8/– ,, ,, ,,
Mass-concrete, class "A" (nominal 1 : 2 : 4)	40/– ,, ,, ,,	40/– ,, ,, ,,
Mass-concrete, class "B" (nominal 1 : 3 : 5)	35/– ,, ,, ,,	35/– ,, ,, ,,
Mass-concrete, class "C" (nominal 1 : 3 : 7)	33/– ,, ,, ,,	33/– ,, ,, ,,
*Concrete in tunnel-linings (class "A")	£3 10*s.* per cubic yard.	£3 7*s.* 6*d.* per cubic yard.
*Reinforced concrete in viaduct-arches, class "R" (nominal 1 : 1½ : 3)	£8 10*s.* per cubic yard.	*Reinforced concrete in columns, beams and slabs, class "R" (nominal 1 : 1½ : 3), £10 per cubic yard.
*Reinforced concrete in viaduct, class "R," in 15-inch to 18-inch interior walls and road-slabs 10 inches thick (nominal 1 : 1½ : 3)	£4 7*s.* per cubic yard.	
Random hammer-dressed masonry, 15 inches thick . .	29/– per cubic yard.	
Squared hammer-dressed masonry facing	12/6 per square yard.	
Dressed granite curb 12 inches by 6 inches with class "B" concrete foundation and backing	14/3 per linear yard.	13/6 per linear yard.
12-inch thick hand-pitched hard-core to road-foundation .	4/7 per square yard.	2/– per square yard.
Steel reinforcement . . .	£14/10 per ton.	£12 to £14 per ton.
4-inch consolidated tarmacadam	5/– per square yard.	4/9 per square yard, 3 inches thick.

* Including all formwork.

Discussion.

The Author. The AUTHOR, after showing a number of lantern-slides illustratin works described in his Paper, said that there were three or fou points which had appeared in the course of the work to which h would like to refer.

Crushed stone of the type of the Penmaenmawr granite was use for aggregate, but if the sand used with it consisted of the fin particles (suitably screened out) resulting from the crushing, it wa necessary to be very careful with the grading, as otherwise ther was a tendency to produce a harsh concrete. An excellent concret could be obtained, however, if care were exercised. It was als necessary to exercise care with regard to the question of dust. Th operation of crushing the stone produced a good deal of dust, whils sand which was crushed stone also contained a good deal of dust It was found as a matter of experience that it was not wise to allov the quantity of dust—by which he meant particles which woul pass a 100 B.S.S. sieve—to exceed 10 per cent., as otherwise th strength of the concrete appeared to be somewhat affected.

Certain of the pre-cast blocks had been found to have soft backs He had found that, when using " Lightning " cement, it was neces sary to pay the utmost attention to curing. When such blocks ha first been made, not enough time had been given to curing, bu when it had been arranged that after being cast the blocks shoul be put into water for 24 hours, the difficulty had been overcome The trouble which had been experienced was exactly as described i Professor Lea's and Dr. Desch's book[1]; the concrete turned brownish-grey and was quite soft.

If he were building another tunnel such as those on the work described, he would take different measures to deal with the questio of leakage through the lining, and the first thing he would do woul be to eliminate construction-joints. The organization would hav to be such that the concreting would be carried straight throug without a stop, or, if that were not possible, he would put in a forr of interlining. If the concreting were carried straight throug without a stop, so as to eliminate construction-joints, he woul expect that there would not be serious trouble with water finding it way through, but, to make it quite secure, he would put a linin inside the concrete. He had learned from his experience with th tunnels described in the Paper that it was asking too much to ex

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[1] "The Chemistry of Cement and Concrete." London, 1935.

The Author.

ɔect that if construction-joints were allowed they would for the most ɔart be watertight. Then there was the question of lighting a unnel of the kind in question. One of the tunnels had electric ights, and it was a curious fact that such a tunnel had to be nade much lighter in daylight than at night, when, in fact, the ights could be turned out. It was rather surprising to observe that ffect.

The Chairman.

The CHAIRMAN, in proposing a vote of thanks to the Author, said hat the Paper described a wonderful piece of engineering, and he ioped that in due course the Author's name would take its place vith those of Thomas Telford, George Stephenson, and Sir Benjamin Baker, who had all been concerned with transport in that district.

Mr. Cook.

Mr. F. C. COOK observed that the Paper described a scheme which vas, as far as he knew, unique in road engineering in Great Britain; t was to some extent similar to such works carried out in Switzerand and in the northern part of Italy. It was perhaps the most pectacular piece of work of its kind which Great Birtain had to how as a result of construction in recent years.

The remains of at least three roads across the headland at Penyclip vere shown in *Fig. 10* (facing p. 43). It was probable that the first oad had been constructed somewhere on the line which was now ɔccupied by the railway, but, as a result of denudation of the coastine and falls of rock from above, the road had, on several occasions, iad to be reconstructed, each time higher up the mountain-side, intil at last it had reached a point about 180 feet above sea-level; urther, it had had a width of only 13 or 14 feet and had been pproached by tortuous approaches on both sides with gradients of ibout 1 in 11. For years traffic on the road had been seriously iindered, and often the lives of road-users had been endangered by alls of rock. The question of whether the responsibility for accidents o road-users lay with the landowners or with the highway authority iad formed the subject of long legal discussions, and the County uthorities had been put to continual expense for the removal of lebris.

The Author had referred to the fact that Sir Benjamin Baker had ɔeen called in to advise on some protective measures for the railway. Sir Benjamin had also been called in in 1899 by the County Council o devise a scheme for a road across the headland; his proposal iad been to site a road as low down the rock-face as was possible, ind to protect it by strong masonry construction. The cost involved vould, however, have been a very considerable sum, and would have ɔeen beyond the financial resources of the County Council in those lays. It was regrettable that until comparatively recent years the naintenance of through traffic along a road of such importance as hat between Chester and Holyhead had been entirely a matter for

Mr. Cook. the local authority, whatever financial position that authority might be in, and it was not until the Ministry of Transport had been set up in 1919 that it had become possible to assist local highway authorities from Government resources.

It was in 1923, as the Author had stated, that a survey had been prepared on the instructions of the Minister of Transport which had resulted in a project for the constructions of both tunnels and a viaduct, and Mr. Cook hoped that that survey had been of some use to the Author when he came to design his scheme. Even at that time, however, the measure of assistance offered by the Government had left such a margin that the local authority could not tackle the work, and it had not been until unemployment was being dealt with seriously, in 1929 and 1930, that the scheme had been allowed to proceed.

The difficulties which the engineers had had to meet in carrying out the work had been great, for below there was the railway tunnel and above there had been the existing road. It had been an essential condition that road traffic should be maintained without serious hindrance, as otherwise a detour of nearly 40 miles would have been necessary for heavy vehicles. He would like to pay his tribute to the engineers and to all those who were responsible for the successful carrying out of a very difficult undertaking without any serious interruption of traffic, and without any untoward happening in the way of preventable accident to the men who were engaged upon the work. It would have been quite easy to devise a scheme which would have been effective in itself, but which would have left a most undesirable scar on what was a picturesque length of coast. The work which had been carried out, however, looked a fitting and integral part of the setting in which it was placed. In particular the portals of the tunnels and the design of the viaduct were excellent examples of engineering appropriate for its setting, and the Author and all those associated with him would have nothing but satisfaction of the success of the work which they had carried out.

Mr. Criswell. Mr. Henry Criswell remarked that, dealing first with the method of survey, it was interesting to note that the Author had adopted the tacheometrical method of survey, which was extremely suitable for the precipitous nature of the terrain. Some years ago, Mr. Criswell had been chief of one of six survey parties in north Iran engaged on the location of a railway over 600 miles long for the Iranian Government, and that method of survey had been adopted over a length of 80 miles, 23 of which passed through a deep gorge composed of limestone and shale. Contours had been plotted at vertical intervals of 10, and in some cases of 5, feet, in order to obtain a "paper" location through the gorge, where shifting the centre-line a few fee

might save large expenditure in rock-cutting. The plan in Figs. 2, Plate 1, showed contours 50 feet apart; it would be of interest if the Author would say whether he had found such an interval sufficient, or whether his detailed surveys had been plotted with contours at closer vertical intervals. Intermediate contours had presumably been omitted merely to avoid confusing the reproduction of the plan. Mr. Criswell.

With regard to the horizontal alignment, the speed for which the road was designed was not stated in the Paper, but the radii of the curves on the Penmaenbach diversion ranged from 350 to 900 feet. On the Penyclip diversion, the radii of curves appeared to range from 500 to 1,200 feet. Were those curves designed for a particular speed or were they designed merely to fit the contours for reasons of economy, irrespective of speed values? He appreciated that on such steep sidelong ground, shifting the centre-line even 2 or 3 feet laterally might effect a great saving in cost. There appeared to be no reference in the Paper to transition-curves, but there was a reference to minimum radius. Did that mean the minimum radius of a wholly-transitional curve? Alternatively, had the Author considered the provision of transition-curves combined with circular arcs, or was the design speed so low as to render transitions unnecessary? On the Penmaenbach diversion there appeared to be one compound curve composed of arcs with a radius of 900 and 700 feet, and also a reverse curve, one leg of which had a radius of 700 feet and the other a radius of 430 feet. The length of straight between the legs of the reverse curve appeared from the plan to be negligible, and it would be of interest to know whether a tangent or a transition-curve was inserted between the reverse curves. If only plain tangential or circular arcs were used, the speed value of the curve was bound to be comparatively low. A point of interest in Figs. 2, Plate 1, was the difference between the alignment of the railway and that of the road. Road engineers, unfortunately, had to adopt much sharper radii and steeper grades than were permitted on a railway.

It would be of interest if the Author would state whether the curves were banked, and, if so, what general rates of banking were adopted and upon what speed the rates were based. The only reference to banking in the Paper appeared to be on p. 64, where it was stated that the effective height of the parapet-wall varied, and depended upon the super-elevation of the carriageway. From that it might be assumed that banking was given on the curves. In view of the importance of correct banking to curves, the subject seemed to merit further reference in the Paper than that given. The reverse curve previously referred to was in special need of banking, and if plain circular arcs were used without transitions or intervening tangent, its execution was bound to have afforded some difficulty

Mr. Criswell. in changing over the slopes of the banking. How was that difficulty overcome?

With regard to the vertical alignment, it would appear that the Author had adopted a standard length of 100 feet for all vertical curves, irrespective of the gradients. Was he satisfied that by that means an adequate length of vision could be assured? On the Penyclip diversion there were two 1-in-20 grades which gave a grade-angle of 10 per cent. The actual vision provided seemed to be 200 feet if the height of the eye were taken as 3 feet 9 inches.

In conclusion, the total length of the works described was slightly over 1 mile, and the cost had been just under £250,000. That mile of road was probably one of the most costly ever constructed, and the figures showed how the advent of the motor-car had revolutionized former ideas of road engineering. A cost of even £50,000 per mile for road construction would have been considered exceptional 30 years ago.

Mr. Hodgson. Mr. G. H. Hodgson said that there were one or two points in the Paper to which he wished to refer. In two places (pp. 39 and 61) the Author referred to "cleavages" in the rock; that hardly seemed the correct term to employ, because it gave the impression that the headlands at Penmaenbach and Penmaenmawr were of a slaty nature, which was quite wrong. The term which Mr. Hodgson thought should have been employed was "joints." Joints were characteristic of all hard and firm rocks, and in the case of the rocks under consideration were most probably due to the shrinkage of the igneous rock on cooling, modified later by great earth movements to which the area was subjected during the Caledonian period. Some of the especially bad places in the rock with which the Author had had to deal were definitely due to faults.

He would also like to say a few words with regard to Table I (p. 66) and also with regard to the Author's remarks about dust (p. 70). The grading of the sand produced from crushed rock was quite different from that of a pit-sand, and a grading suitable for a pit-sand was most unsuitable for what he would call a quarry-sand. For a quarry-sand, all materials should pass a $\frac{3}{16}$-inch mesh, from 20 to 25 per cent. should pass a 50 B.S.S. mesh, and from 10 to 12 per cent. a 200 B.S.S. mesh. Mr. Hodgson then showed some lantern-slides comprising photomicrographs of various fine powders, sands and thin sections of concretes. Limestone dust which had passed through a 200-mesh sieve was seen to be an impalpable powder, whilst Penmaenmawr stone dust which had passed through a similar sieve was a very coarsely crystalline powder. Stone-dust, provided that it was chemically stable, was a coarsely crystalline powder, and was relatively large compared to the cement-particles. A percentage of that dust in a concrete mixture was very beneficial,

as it added to its strength properties and workability, and helped to make the concrete watertight, so giving additional protection to the steel reinforcement. It also reduced the moisture movement of the finished concrete. Tests with percentages up to 40 per cent. of dust in the sand-content showed no loss of crushing strength. With the introduction of 10 per cent. of stone-dust into a 3 : 1 mortar composed of cement and pit-sand, with round and smooth particles, the strength was increased by 100 per cent. Mr. Hodgson.

The photomicrographs of thin sections of 6 : 1 concrete showed, in the case of the mortar composed of pit-sand, that the round and smooth sand-particles were coated with cement, and that the relatively large void-space between the sand-particles had to be filled with neat cement. In the case of the mortar composed of quarry-sand, containing about 10 per cent. of stone-dust, it was seen that all the particles had a rough fracture, and were coated with cement, whilst the little particles of dust had acted as "plums," helping the cement to fill the void-space between the sand-particles. The rough fracture would add anything up to 100 per cent. in toughness of the concrete, compared with concrete made with certain smooth and rounded aggregates.

It was often said that having dust in the aggregate meant that it was necessary to use more water for mixing the concrete, but he wished to make another suggestion. The surface-area of the aggregate was increased to a large extent if it had a rough fracture. He would imagine that when water was added to a dry mixture of cement, fine and coarse aggregate, and before any chemical action between the cement and water had commenced, that each minute particle of cement was surrounded with a film of water, and that all the particles composing the fine and coarse aggregate would also have to be wetted. An aggregate with a rough fracture and therefore having a larger surface-area to be wetted would, therefore, require more water. The need for extra water was often attributed to dust, when it had nothing at all to do with dust, but was due solely to the rough fracture. Again, concrete made with a quarry-sand without dust was harsh and unworkable, but with 10 per cent. of dust in the sand the concrete was readily workable.

Mr. W. T. Halcrow remarked that his connexion with the scheme had entailed two visits to the works described. He had asked the Author why the road had been built along the face of a difficult hill-side, and whether it would not have been possible to have driven a long tunnel through the hill and thus to have avoided the necessity of building many large retaining walls and other heavy structures. The Author had explained to him at the time that such a scheme had been considered, and the reasons why it had not been adopted were given in the Paper. Did the Author still hold Mr. Halcrow.

Mr. Halcrow. the opinion that the road along the face of the cliff was the better scheme, when considered in the light of the experience gained in carrying out the work?

Reference was made (p. 36) to the deep cuttings in the rock, the slopes of which were designed for a batter of 8 to 1. He thought that that batter was very difficult to obtain in rock, and he usually adopted a slope of 4 to 1. The face of the rock had apparently been protected in rather a patchwork manner; that was to say, only those areas on the face of the cliff which showed signs of possible falls of rock at a later date were faced with masonry, and that seemed to detract somewhat from the general appearance of the road. If a batter of 4 to 1 had been adopted, with a full facing of masonry, the cost might not have been increased to any appreciable extent, the appearance would have been improved, and any possibility of further facing work, where the rock might possibly become loose through the action of weather, would have been avoided.

The Author, when laying down the method of driving the tunnels, had required a pilot-heading to be located near the top of the tunnel; Mr. Halcrow suggested that it would have been better to have located the pilot-tunnel at road-level, and between the centre-line and the high side of the hill, so that the pilot-tunnel would have been driven in the best rock available. The effect of shot-firing in the confined space of a pilot-tunnel was more severe than that of blasting down to the full section from a previously-driven pilot-tunnel. It appeared to him that the rock would have been sound enough in the position which he had suggested to drive the pilot-tunnel without any timbering, except perhaps at the ends.

He hardly liked to make a criticism of the viaduct, which was a very beautiful structure, but he believed that it could not be seen from the road; it could only be seen from the railway, by passengers who were perhaps travelling at the rate of 50 miles an hour. The piers and the arches, however, were all faced with pre-cast concrete blocks, and it was originally intended that they should have been granite. Was that really necessary, and could not the work have been made as effective in appearance at possibly a lower cost, if it had been constructed in masswork?

Mr. Frank. Mr. T. Peirson Frank observed that, on the viaduct (Figs. 22, Plate 2), the walls carrying the roadway were placed transversely instead of longitudinally. Was there some special reason for that unusual arrangement?

In the course of the discussion it had been mentioned that Penmaenmawr headland had long been an obstruction to transport, but the stone from the locality was of the greatest possible assistance to transport. In fact, for the heaviest type of traffic with which he was acquainted, which was to be found in the Liverpool dock area,

the paving stone had to come from one of two quarries in the Penmaenmawr district if reasonable life were to be obtained. Mr. Frank.

He believed that the Author was correct in suggesting a continuous lining for the tunnels, although there were some who would criticize the proposal. Was the Author's reason for making that suggestion for future work the fact that in such a position extremes of temperature which caused rather severe contraction-cracks did not occur? Mr. Frank believed that in works where there was equable temperature a certain amount of jointing could be omitted. It was the custom in the north of England, and particularly in the north-west, to omit many construction-joints in reinforced-concrete work, but when that practice had been suggested for adoption in the south—the suggestion had been that surface-concrete for roadways should be made in continuous lengths—it had not been universally approved. There were, however, examples in Surrey, carried out by a Committee over which Mr. F. C. Cook presided, of what were termed continuous lengths of construction, in which no cracking, or at any rate no appreciable cracking, had occurred.

He agreed with what the Author had said with regard to the curing of rapid-hardening cement. In testing that material against a number of other cements in a warm locality in the extreme south-west of England, he had found that the only case in which what might be termed hair-cracks in surface-construction had occurred had been where rapid-hardening cement had been used.

Mr. Halcrow had referred to the position of the pilot-tunnel. Mr. Frank did not know the locality, and he did not know whether there was much subsoil water in the ground, but there appeared to be large crevices or cracks between sections of the rock; were those crevices water-bearing? If there was much water there, probably a pilot-tunnel at the foot of the subsequent tunnel might have acted in part as an effective drain. He believed that such a pilot-tunnel had been constructed in the case of the Mersey tunnel, and also in other cases, but there were probably good reasons for locating the pilot-tunnel in the upper portion of the rock in the works described in the Paper; for one thing, for example, that position might have facilitated excavation.

*** Mr. E. G. WALKER observed that both sections of the work involved tunnelling, and that on the Penmaenbach section the tunnelling appeared to have been the principal item of cost in the works. It was, therefore, a little disappointing that no information was given as to the methods adopted in the tunnel excavation beyond two short notes on pp. 39 and 40; a fuller description of the enlargement of the heading and of the placing of the lining would be of advantage. Mr. Walker.

*** This contribution was submitted in writing.—SEC. INST. C.E.

Mr. Walker. As it was stated that there was a considerable amount of fissured and unstable rock on the site, it would be of interest to learn whether any special precautions had had to be taken in the tunnel to support faulty places in the linings. Some description of the concreting methods would also be of use.

He wished to have more information about the tests on the effect of blasting on concrete, referred to on pp. 40 and 41. There was no reference to the 6-inch cubes and percolation-test specimens having been cut out of concrete already deposited, so that it was assumed that those specimens were simply placed in position and tested after they had been left exposed to any effect that might have been produced by the blast. If that were correct there did not seem much point in making such a test, because the effect of a blast upon green concrete or upon concrete that had only just set could surely only be ascertained by taking specimens from concrete that had already been placed in position and therefore subjected to some measure to the restraints imposed by its surroundings. Some further elucidation of the nature of the tests was necessary in order to be able to appraise their value properly.

The Author. The AUTHOR, in reply, said that, with regard to the remarks made by Mr. Cook, he would like to pay tribute to the value of the original survey made under the order of the Minister of Transport. That had been of the greatest possible use in the preparation of the first report on the scheme. Mr. Cook also referred to the portals of the tunnel, and the Author felt that it was only right that it should be made clear that the design of the Penmaenbach portals was made by Mr. H. Chalton Bradshaw, the present Secretary of the Royal Fine Arts Commission.

Mr. Criswell had asked for information about the contour-intervals and the speed for which the road was designed. The contour-interval at Penmaenbach was 10 feet, and at Penyclip it was 5 feet. No special speed had been considered. The radii of the curves had been settled more by the question of a reasonably economical layout than by any other consideration, and as the excavation was in solid rock it was clearly desirable to keep it to a minimum quantity. It was not considered necessary to utilize transition-curves. The Author felt that a relatively clear distinction ought to be drawn between railway practice and road practice in such matters as the lay-out of curves, and that many criteria which applied to the proper construction of a railway track could be neglected in road practice without any detriment to the result. The curves of the roads were banked and a cross-fall of 1 in 40 was adopted. Whilst the Author agreed that in general greater superelevation was advantageous, in the case under discussion, owing to the fact that rather sharp reverse curves were essential, it was not practicable to arrange the necessary

hange-over in the banking with greater superelevation than that adopted. Actually the changing-over was carried out in a length of about 80 feet for each reverse, although the length of straight between tangent-points was in most cases considerably less. With egard to vertical curves, it was found in practice that the arrangenent adopted—that was, a standard length of 100 feet for each ertical curve—gave a very good length of vision having regard to he general nature of the road. If the road had been across open nd more or less featureless country and, therefore, one on which igh speed might be attained, a longer line of vision would have een necessary. The Author.

With regard to the word "cleavages" which was mentioned by Ir. Hodgson, the Author agreed that the word "joints" was to be referred, because the Penmaenmawr rock was hard and firm.

The Author agreed that dust helped to make a workable concrete, ut the fact was that the tests throughout the works showed that, f the percentage of dust was allowed to become excessive, the trength of the concrete fell off.

A longer tunnel could have been driven in order to save so many eavy retaining walls, but the preliminary estimate showed that it vould have been materially more expensive. The County Council referred the present arrangement from the point of view of appearnce and, in the light of the final result, the Author considered that he County Council were right. Had the slopes been taken out at to 1, they would have encroached on the existing road and would ave rendered it impossible to keep the latter open to traffic. The ace of the rock in the cuttings was protected in rather a patchwork nanner because, in the beginning, it was not intended to face those uttings. He was doubtful, however, whether a uniform facing arried right through would have improved the appearance of the vork as, although the present facing was patchwork, the Author hought that it suited the nature of the countryside fairly well. 'urther, a continuous facing would have added considerably to the ost of the work.

In regard to the position of the pilot-heading, the view which was eld by the Author was that the nature of the rock was such that, there had been a bottom heading, the process of opening out ould inevitably have let the situation get out of control in places, nd would have greatly increased the risk of bringing down the xisting road, which was above the tunnels.

The viaduct was actually faced with hammer-dressed Penaenmawr stone; that method of construction was quite economical ecause no shuttering was required for the concrete hearting, and e effect was good.

The walls which carried the roadway on the viaduct were placed

The Author. transversely from consideration of the design. The loading was then known to be applied at definite points on the barrel of the arch. Had longitudinal walls been used the distribution of the loading on the barrel would have been undeterminate, and whilst the longitudinal walls would no doubt have given some stiffening effect, it was doubtful whether any reduction could have been made in the thickness of the barrel. In any case, the expansion-joints over the piers would have broken the continuity of those longitudinal walls.

With regard to the lining of the tunnel, the view of the Author was that in such a position extremes of temperature did not occur, and that contraction-cracking from that cause was virtually non-existent.

It ought to be explained that in regard to the quick-setting cement for pre-cast blocks, the Author's remarks were applied to high-alumina cements and not to rapid-hardening cements.

There was a certain amount of subsoil water in the ground, but it was relatively trifling in amount and was under no head.

The method used in tunnel-excavation was first to drive a top pilot-heading and then, in the more shaky rock, to divide the face into a number of pillars which were taken out in a definite sequence. That limited the possible effects of any collapse and enabled the work to be done safely. The lining was placed by the use of steel centres and timber shutter-boards. The first concrete placed was in the side walls up to the springing of the arch, and those walls were done ahead of the arch. The arch followed on in lengths which varied somewhat, the concrete being placed from the sides upwards.

A good many precautionary measures had to be taken with rega to supporting unstable rock. For example, a large fissure l appeared above the pilot-heading of the eastern tunnel at Penyc and was held by the use of steel bars and concrete. Buttress was done at the portals of some of the tunnels in order to supp faulty rock. A description of that work was given on pp. 60 and

It was not possible to make a test of the type suggested Mr. Walker because the green concrete actually placed for walls was not sufficiently close to the face of the blasting. Obviou it could not be allowed to approach within 20 feet, and theref special specimens had to be used so that they could be put close the blasting. The object of those tests was merely to take extreme case and to see if there were any definite evidence of dam

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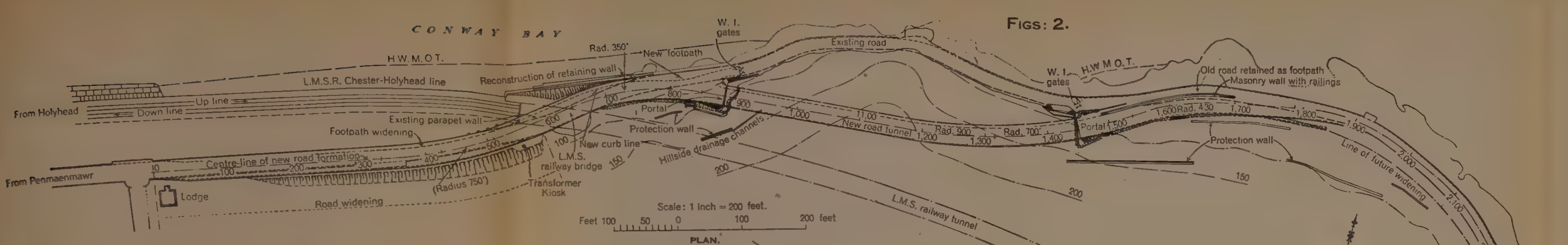
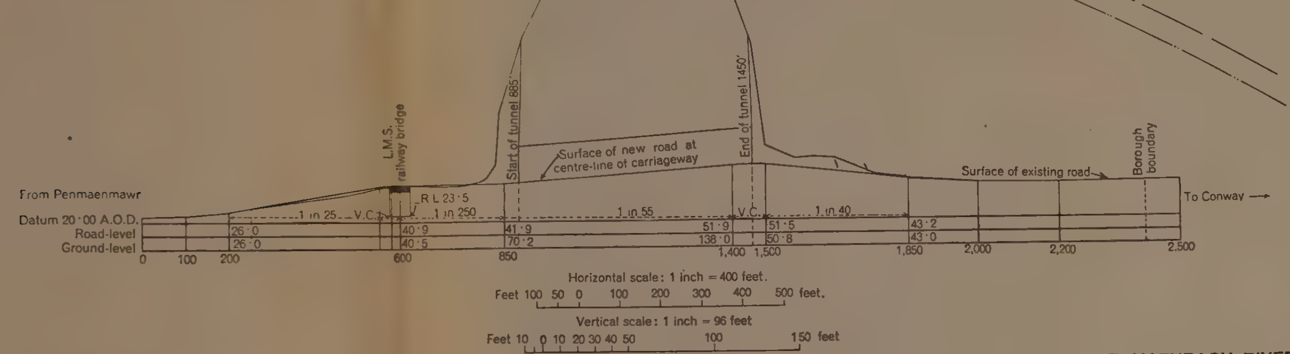
*** The Correspondence on the foregoing Paper will be publis in the Institution Journal for October, 1937.—SEC. INST. C.E.

THE RECONSTRUCTION OF THE CHESTER–HOLYHEAD ROAD NEAR PENMAENMAWR, NORTH WALES.

FIGS: 2.



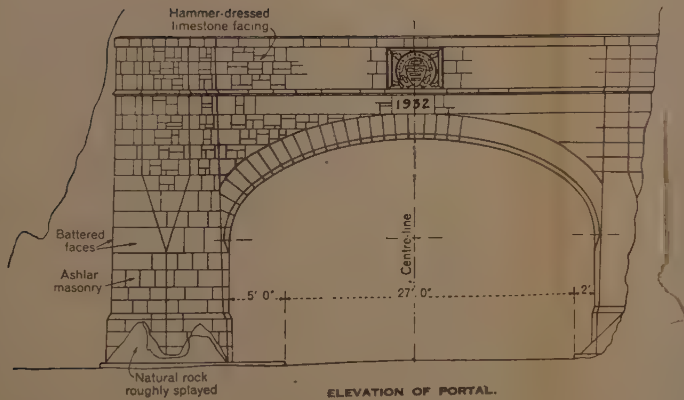
PLAN.



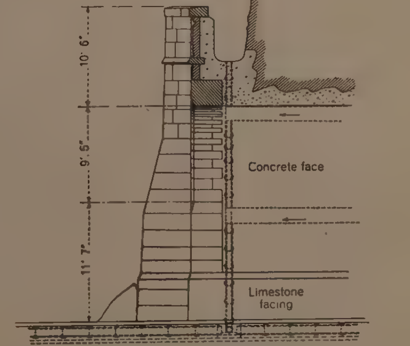
LONGITUDINAL SECTION.

PENMAENBACH DIVERSION.

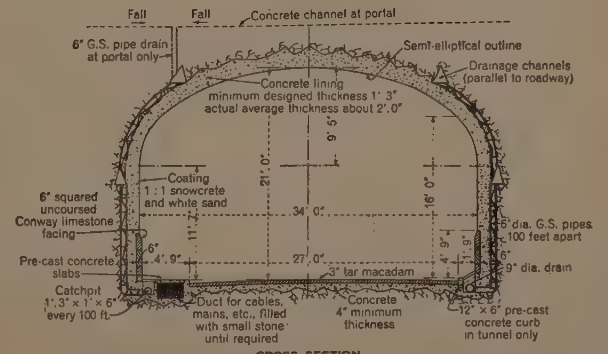
FIGS: 8.



ELEVATION OF PORTAL.



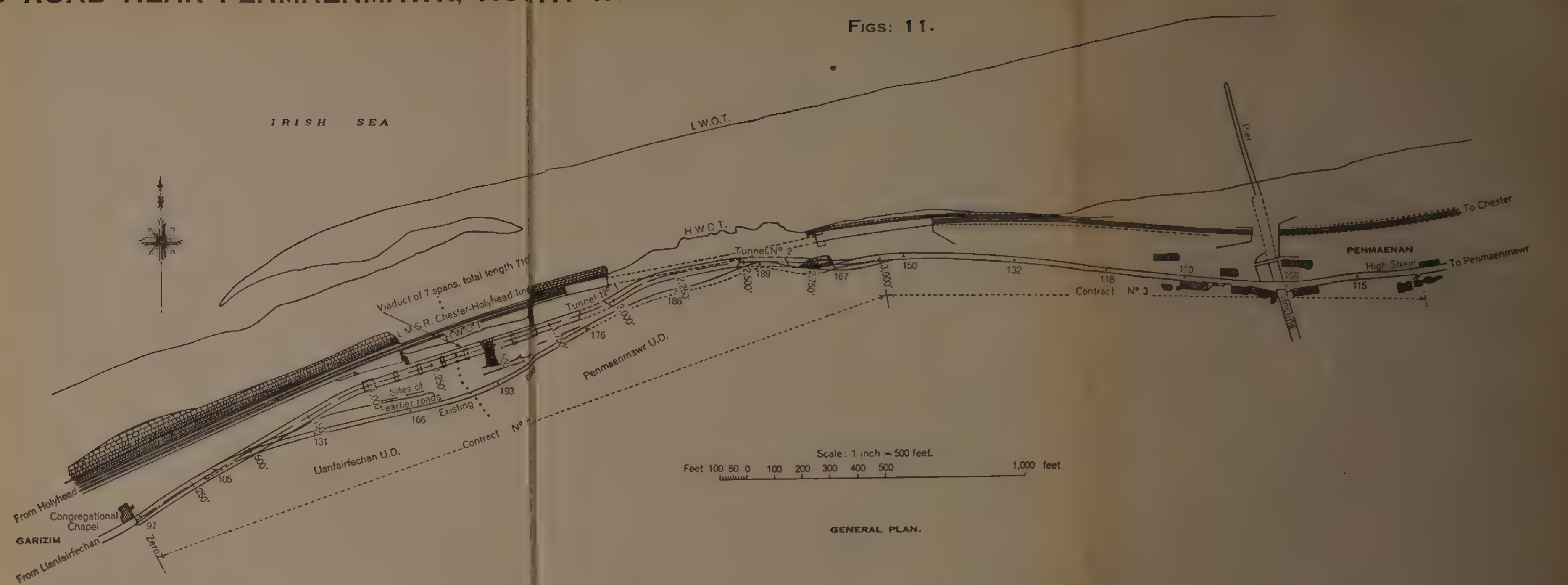
LONGITUDINAL SECTION.



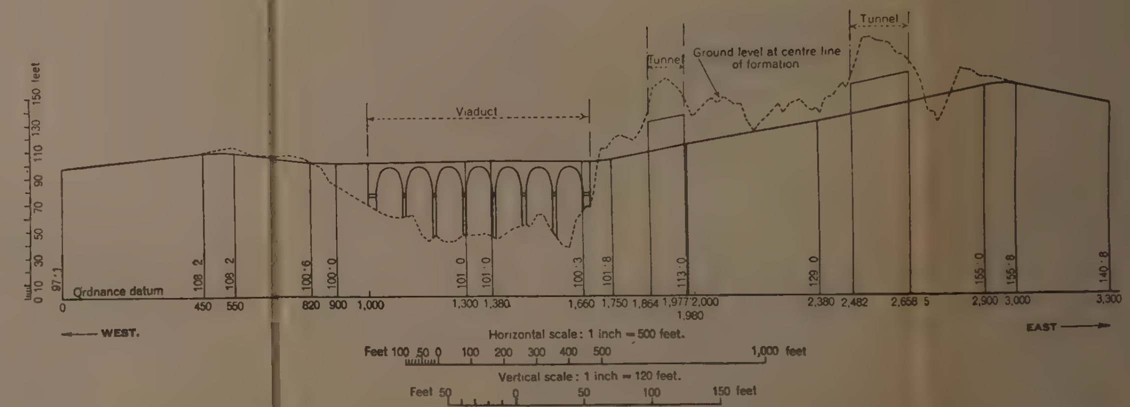
CROSS-SECTION

DETAILS OF PENMAENBACH TUNNEL.

FIGS: 11.



GENERAL PLAN.



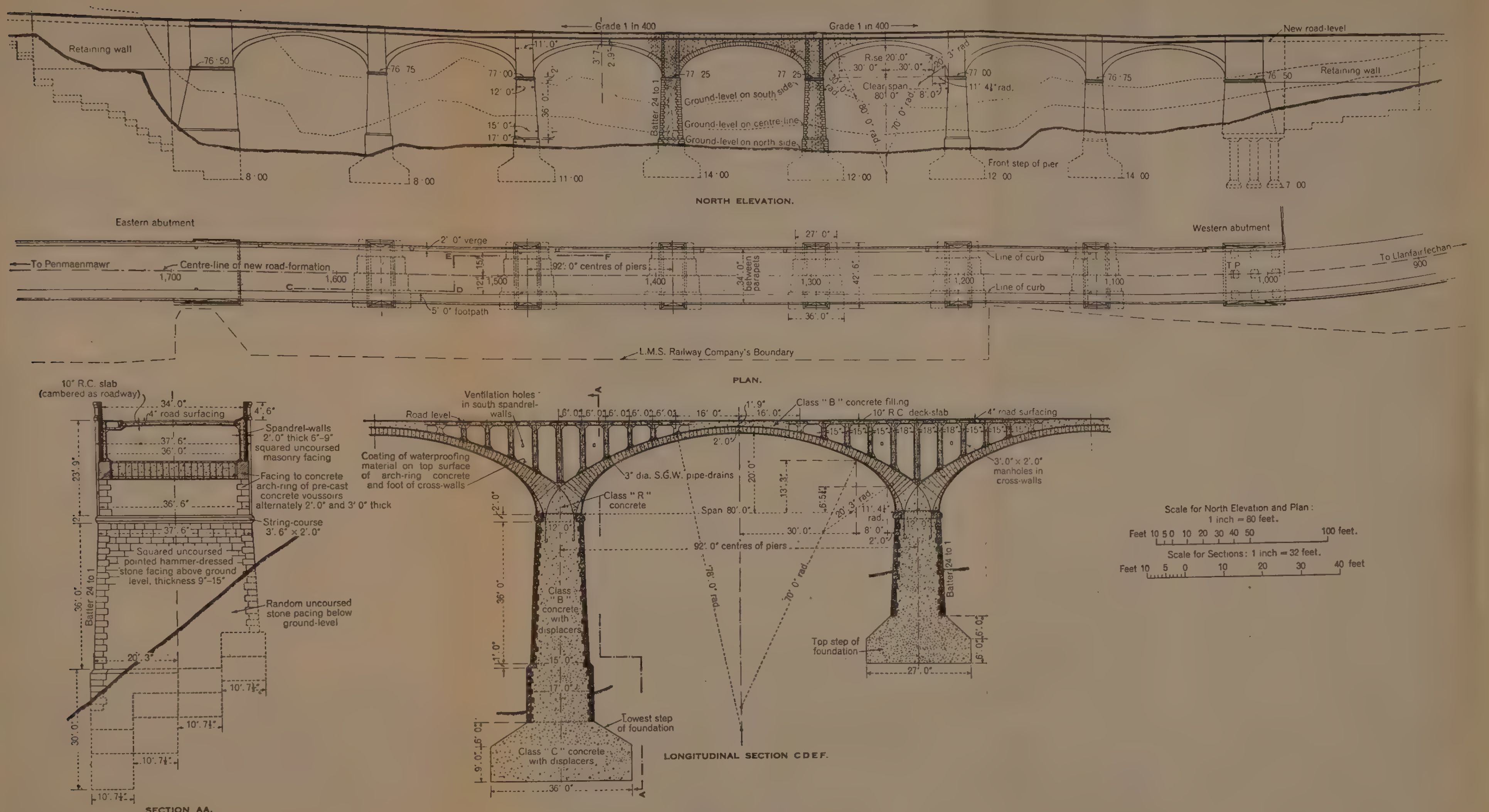
LONGITUDINAL SECTION.

PENYCLIP DIVERSION.

C. L. H. HUMPHREYS.

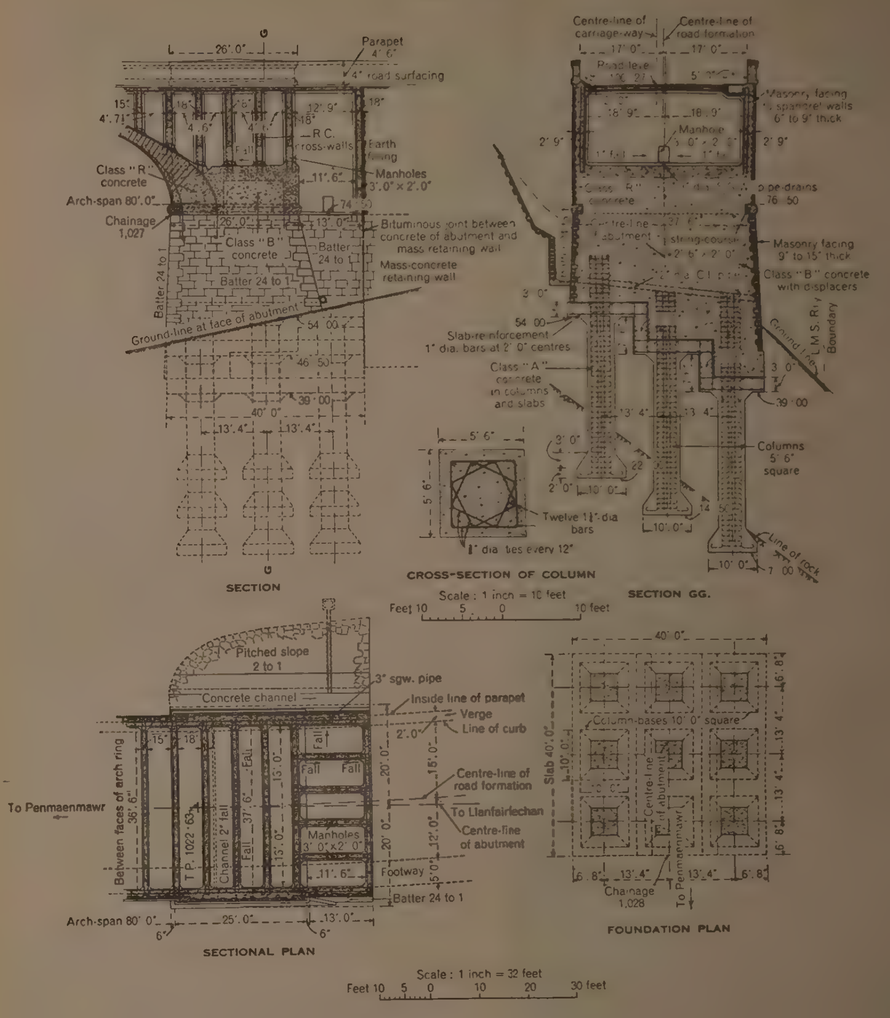
THE RECONSTRUCTION OF THE CHESTER—HOLYHEAD ROAD NEAR PENMAENMAWR, NORTH WALES.

Figs: 22.



PENYCLIP VIADUCT.

Figs: 23.



DETAILS OF WESTERN ABUTMENT.

C. L. H. HUMPHREYS.

WILLIAM CLOWES & SONS, LIMITED: LONDON.

ORDINARY MEETING.

20 April, 1937.

SYDNEY BRYAN DONKIN, Vice-President, in the Chair.

It was resolved—That Messrs. F. H. Brunt, S. W. Budd, G. T. Cotterell, J. D. C. Couper, George Evetts, A. J. Martin, R. E. Tickell and P. J. H. Unna be appointed to act as Scrutineers, in accordance with the By-laws, of the ballot for the election of the Council for the year 1937-38.

The following Paper was submitted for discussion, and, on the motion of the Chairman, the thanks of The Institution were accorded to the Authors.

Paper No. 5113.

"The Flow of the River Severn, 1921–36."[1]

By Professor STEPHEN MITCHEL DIXON, O.B.E., M.A., B.A.I., M. Inst. C.E., GERALD FITZGIBBON, B.A., B.A.I., and MICHAEL ANTHONY HOGAN, D.Sc., Ph.D., M. Inst. C.E.

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INTRODUCTION.

THE lack of information regarding the flow of rivers in Great Britain was commented upon in the Second Interim Report of the Water Power Resources Committee of the Board of Trade.[2] With regard to the work of the proposed Commission, which was to have charge of the resources, it was remarked that the existing sources of information would need supplementing in certain directions; the Report

[1] Correspondence on this Paper can be accepted up to the 1st September, 1937.—SEC. INST. C.E.

[2] Cmd. 776, 1920.

continued, " This is particularly the case in regard to the quantity of water available in the various catchment areas in this country owing to the almost complete absence of river gaugings."

At that time there was some doubt as to the best method of obtaining records of river-flow, and the present investigation was begun with the twofold object of studying the methods of measuring and recording discharge, and of accumulating information with regard to the variations in discharge and their relation to the rainfall. Soon after the work had begun, the Department of Scientific and Industrial Research accepted the financial responsibility for that portion of the investigation which dealt with the methods of discharge-measurement, and the supervision of the work came under the control of the Department's Committee on the Gauging of Rivers and Tidal Currents. This arrangement lasted from June, 1921, to the end of 1923.

The Severn was selected because it was the only large river free from artificial controls and river-traffic within convenient distance from London. When the records were commenced it was not considered likely that the records of discharge at Bewdley would, in themselves, be of much economic importance, and the work was regarded rather from a scientific point of view. Recently, however, a Catchment Board has been set up for the basin of the Severn, in accordance with the Land Drainage Act of 1930, and the information gained at Bewdley should prove to be a useful foundation for the Board's studies of the flow of the Severn. In 1936 the Catchment Board decided to undertake the maintenance of the records at Bewdley which form the subject of this Paper, and it seems to the Authors that this is a suitable occasion to review the results obtained during the past 15 years.

Description of the Catchment.

The portion of the Severn valley dealt with in this investigation lies above Bewdley and has an area of 1,632 square miles (*Fig. 1*). The river rises on the eastern slopes of Plynlimmon at a height of over 2,000 feet above sea-level, and flows first in an easterly direction. It then takes a sharp turn northwards to Welshpool and emerges on to a flat plain, over which the course is at first east and then south through Shrewsbury. On the plain above Shrewsbury the slope is very slight and the river has a very meandering course. At Ironbridge the character of the river undergoes a change and it flows through a narrow rock channel with a fairly steep slope. Below Ironbridge the channel widens and the slope diminishes again until the river reaches another rocky section from Bridgnorth to Bewdley.

The mountains forming the western boundary of the catchment

and the highlands on the southern boundary consist of Palæozoic rocks which may be considered to be relatively impermeable. The region to the north and east is largely composed of Triassic strata

Fig. 1.

AVERAGE ANNUAL RAINFALL IN THE SEVERN VALLEY, 1881–1915.

overlain by glacial drift, which are probably very pervious. The northern boundary of the catchment is situated in a low-lying drift-covered region, with many bogs and small lakes, which drains on the north to the Dee and Weaver. The present course of the

river seems to be the result of glacial action, and there are indications that the pre-glacial valley ran northwards and is now covered with drift. It seems possible that a considerable quantity of water may be lost to the river by percolation along the direction of its old channel, particularly at low stages of flow.

The works for the water-supply of Liverpool impound a catchment-area of 23,500 acres on the headwaters of the Vyrnwy,[1] one of the principal tributaries of the Severn. The area above the Vyrnwy dam is 2¼ per cent. of the total catchment to Bewdley. The quantity of water sent out of the Severn catchment to Liverpool varied from about 17 to 19 cusecs during the period of the records. Besides the water used for supply, there is a further loss by evaporation from the surface of the lake (1,121 acres in extent). It is estimated that the effect of the works on the total annual flow at Bewdley is equivalent to a loss of 0·15 inch of rainfall, or less than 1 per cent. of the total. Compensation-water at the rate of 10 million gallons per day (18·6 cusecs) is discharged from the reservoir, and there is provision for periodical flushing at a rate of 40 million gallons per day (74·4 cusecs) for 4 consecutive days when required. The effect of the Vyrnwy works on the flow at Bewdley is very difficult to estimate; when the water is below the crest of the dam the only flow passing the Vyrnwy dam is the compensation-water, but with a full reservoir the natural flow of the stream less the quantity sent to Liverpool and the loss on the surface of the lake is discharged.

Selection of the Gauge-Points and Gauging-Section.

After preliminary surveys and gaugings had been made at Bridgnorth, Arley, and Bewdley, it was decided to establish a gauge at the last place because it was the most easily accessible. A prominent ledge of rock ran obliquely across the river close to the school about 200 yards below Bewdley bridge, and it was considered that this would form a satisfactory control. A gauge-scale, at first of timber and subsequently of cast iron, was fastened to the quay-wall about 50 yards below the bridge. The water-level on this gauge has been read daily since March, 1921. Another gauge was established at the Elan Aqueduct bridge of the Birmingham Corporation Water Department in June, 1921. This is about 2½ miles upstream from Bewdley, and no tributary of any importance enters between the two gauges. The area above the Elan Aqueduct gauge is 98·8 per cent. of that above the Bewdley gauge. An automatic water-level recorder was installed at the Elan Aqueduct bridge in June, 1923.

The gauging-section used for discharge-measurement is situated

[1] The dam was completed and impounding commenced in November, 1887.

about 1,000 yards above Bewdley bridge in a straight and uniform reach with a relatively smooth rock bottom. The site was selected after the river had been carefully surveyed at low water during the dry summer of 1921. The gauging-section is in the centre of a straight and uniform portion of the channel about 300 yards long. Above this there are slight rapids which are drowned when the flow exceeds about 3,000 cusecs.

Methods of Discharge-Measurement.

As one of the objects of this research was to investigate the relative advantages of different methods of determining the discharge, a greater variety of methods was employed than if the discharge of the river were the only subject being studied. This part of the research has already been described in detail [1] and reference should be made to that description for a fuller account of the methods of discharge-measurement.

Gaugings were usually made with current-meters, but surface-float observations were also made. In the current-meter measurements the velocity of the water was measured on a series of verticals 10 feet apart across the section, and observations were taken on each vertical at points at different depths, usually 1 foot apart. Curves showing the variations in velocity with depth were plotted for each vertical, as shown in the lower part of *Fig. 2* (p. 86). The area of a vertical velocity-curve is equal to the discharge per unit width at that vertical. By plotting these unit discharges at their verticals, a curve is obtained showing the distribution of the discharge across the section (plotted above the water-line in *Fig. 2*) and the area enclosed by this curve is equal to the total discharge. The details of a typical discharge-measurement, made on the 28th January, 1927, are given in *Fig. 2*. The water-level at the gauging-section was 13·1 feet on the scale, and the discharge as measured with the Price meter was 7,060 cusecs. To avoid undue complication, only alternate vertical velocity-curves are shown on the diagram.

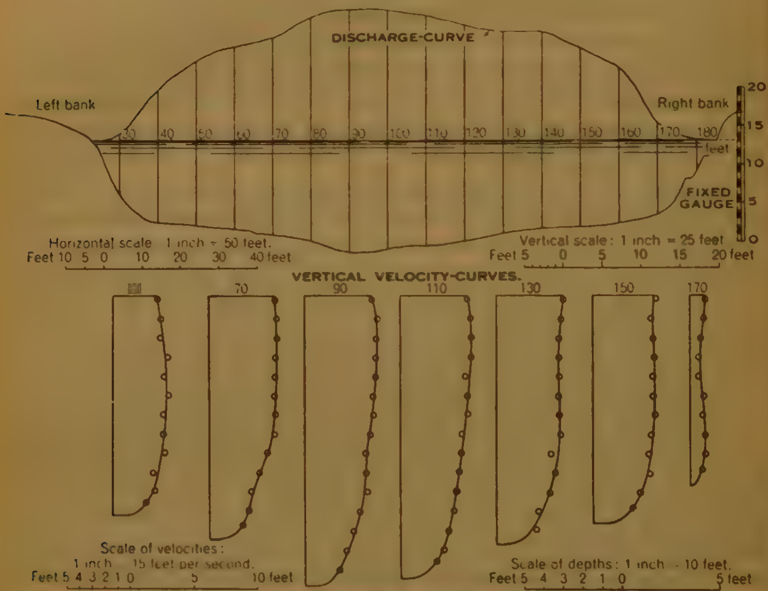
It will be appreciated that the methods used for gauging must be chosen to suit the characteristics of the river under investigation. The Severn at Bewdley changes stage relatively slowly, the maximum alteration in water-level during any gauging being 0·3 foot, and thus it was unnecessary to take special precautions against change of stage in the course of the individual measurements.

The current-meters used in the gaugings were all fitted with electrical contacts to indicate the passage of one or more revolutions. It was found that a long interval between individual signals was

[1] M. A. Hogan, "River Gauging," Department of Scientific and Industrial Research, London. H.M. Stationery Office, 1925.

undesirable and caused much loss of time, better results being obtained when the meter gave a signal every 5 or 10 seconds. As a rule the revolutions of the meter were timed over a period of about a minute. The meters were of two types, the rotating-bucket type represented by a small Price meter, and the propeller or screw type represented by an Amsler 1915 type, an Ott type V and a Stoppani meter. The Price and Ott meters could be used either on a rod or freely suspended from a cable, but the Amsler and Stoppani

Fig. 2.



Results of a Current-Meter Discharge-Measurement.

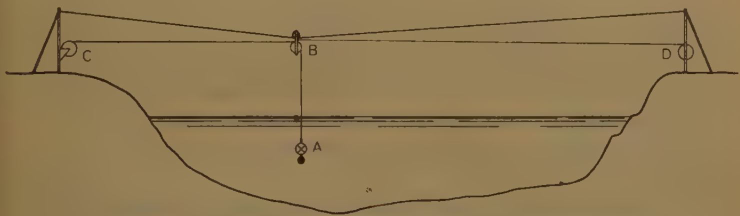
meters could only be used on a rod. Three methods of supporting the meter can be distinguished as follows:

(1) *Cable suspension.*—The majority of the measurements used in connexion with the maintenance of the records of discharge were made with a Price meter suspended from a cable, but other methods were used for experimental and instructional purposes. A cable suspension-system for operating the meter was originally developed for use at times of high water when the use of a boat was impracticable, but was found to be so convenient that it was subsequently employed for all the gaugings made with the meter suspended (*Fig. 3*). Steel posts were permanently fixed on either side of the section and from these a suspension-cable was hung whenever

a gauging was to be made. A simple traveller running along this cable served to carry the meter. The two control wires to fix the position of the meter horizontally and vertically on the section were operated from opposite banks, thus avoiding trouble due to tangling. The traveller winch which controlled the horizontal position of the meter was on the left bank, and the measurements of the horizontal distance across the section were made by the aid of graduations on the stranded wire which held the traveller. This winch was locked in position while a series of observations were taken on the vertical.

The vertical position of the meter was controlled by a wire cable running over a pulley on the traveller to a winch on the right bank. At each vertical, the work was begun by taking a sounding; the bottom of the meter-weight was lowered until it just touched the water surface, the end of a measuring tape was clipped to the wire cable and the drum slacked off until the weight touched bottom,

Fig. 3.



(A, meter; B, traveller; C, meter-winch; D, traveller-winch.)

CABLE SUSPENSION-GEAR FOR OPERATING CURRENT-METER.

when the reading on the tape gave the depth. It should be remarked that no difficulty was experienced in determining when the meter-weight touched the bottom, as the pull on the winch fell off and the main cable changed its shape owing to the release of tension. Soundings could be made in this way to within 0·1 foot. The position of the meter could then be adjusted by reference to the tape for taking observations of velocity at different distances below the surface.

This method of sounding suffers, however, from a systematic error due to the effect of the velocity of the water in dragging the meter and cable downstream. The problem of correcting soundings made under such conditions has been discussed by Dr. Percy Phillips,[1] but on the Severn it was possible to use simpler methods. As the shape of the section remained unchanged throughout the period of the records the amount of error could be determined by comparing

[1] Egypt, Public Works Ministry, Physical Department Paper No. 18. Cairo, 1924.

the meter sounding with the true depth given by the difference in level between the water-surface and the bottom at each vertical. The maximum error noted, using a small Price meter with a 30-lb. torpedo-shaped weight, was 7 per cent. when the depth was 15·7 feet and the mean velocity on the vertical was 5·25 feet per second. In the majority of the gaugings the error due to meter-drag was negligible, but allowance was made for it in all measurements exceeding 3,000 cusecs.

A few gaugings were made with an Ott propeller-type meter suspended from this gear.

(2) *Standing rod.*—The first measurements on the Severn were made with an Amsler propeller-type meter. This was fixed to a 1-inch diameter rod made of gas-pipe and provided with a foot-plate and arm for the attachment of a stay-wire. The rod was lowered on to the bottom and held vertically at the side of a 20-foot punt, the stay-wire being used when the depth exceeded about 7 feet. The punt was held fore and aft by stranded steel cables 3/16-inch in diameter spanning the river, and the position of the observation was determined by reference to a tape suspended from one of these cables. With this arrangement traverses were made across the river, taking observations at points 10 feet apart with the meter at definite distances above the bottom. The meter was left at a fixed depth throughout each traverse because it was inconvenient to slack off and shift the meter on the rod several times at each vertical to obtain the points for a vertical velocity-curve. This method involved a great deal of moving of the punt but the meter had only to be shifted on the rod after each traverse.

The standing-rod method was used at Bewdley in order to show students a method of gauging in which the meter was the only special apparatus required. For routine use the cable-suspension method was preferable because it could be worked by two observers, whereas five persons were usually required in the punt.

(3) *Suspended rod.*—This method requires the provision of a steady platform from which the meter, attached to the end of a rod, can be operated. Only one measurement was made by this method at Bewdley. The suspended-rod type of mounting has the advantage that the depth of the meter can readily be varied so that the observations for a vertical velocity-curve can rapidly be completed at each station. The necessity for a stable platform renders this method more expensive than the cable-suspension, at least on rivers of moderate width.

The Accuracy of the Discharge-Measurements.

The two quantities measured in a gauging are area and velocity. In the present series of gaugings the area of the section was measured with a high degree of accuracy, owing to the presence of a rocky floor which enabled the soundings to be checked and corrected as already described. The only slight variations in the shape of the section were caused by slumping of material from the banks. As this material was only deposited in the comparatively slow-moving water at either end of the section its influence on the discharge would be negligible.

The accuracy of the velocity-measurements depends on the type of meter and method of support, on the number of points at which observations are taken, and on the duration of the observation at each point.

The two types of meter used are differently affected by the oblique velocities characteristic of turbulent flow. The cup-type (Price) meter, with buckets rotating in a horizontal plane, responds to velocity-pulses irrespective of the direction from which they come, whereas the screw-type (Amsler or Ott) meter, with a horizontal propeller, only records a reduced component of uniaxial velocities. Consequently it would be anticipated that under conditions of turbulent flow the cup-type results should be high and the screw-type results low. Simultaneous or nearly simultaneous measurements were made with the two types of meter on sixteen occasions under the ordinary conditions of use, the cup type being freely suspended, the screw type on a standing rod. When adjustments have been made for slight differences in the mean water-level of the individual pairs of gaugings, it appears that on twelve occasions the discharge given by the cup-type meter was greater than that given by the screw-type meter, the average difference being 2·3 per cent. On one occasion the two discharges were equal, and on three occasions the screw-type meter discharge was greater than that of the cup type, the average difference being 1·2 per cent. Taking the average of the sixteen pairs, the cup-type meter discharge was greater than that of the screw-type meter by 1·5 per cent. The true discharge will lie between the values given by the two types of meter, and it may be concluded that the average systematic error due to the type of meter is less than 1 per cent., so that either screw or cup type will give results which are sufficiently accurate.

The accuracy of the discharge-measurements may also be considered in relation to the stage–discharge curve, but it must be noted that the stage–discharge relation may vary from time to time

owing to changes in the control or to rapid changes in stage. The stage–discharge relation at the gauging-section remained very constant throughout the period 1925–1933, when frequent measurements were made, and the average departure of the points for the individual discharge-measurements from the mean curve is about 3 per cent. Of the fifty-nine gaugings during this period, twenty-eight showed a departure less than 2·0 per cent., forty-five a departure less than 5 per cent., and only one a departure greater than 10 per cent. from the mean curve. From this it is concluded that the stage–discharge curve will, on the average, give the discharge within ± 3 per cent.

It will be noted that the average departure of an individual discharge-measurement from the mean curve, namely 3 per cent., is twice the average difference between a pair of simultaneous measurements by two different methods, 1½ per cent. The latter figure is probably a better indication of the precision of the discharge-measurements, and the larger departure from the stage–discharge curve must be attributed to the effect of the above-mentioned factors, which are not connected with the discharge-measurement.

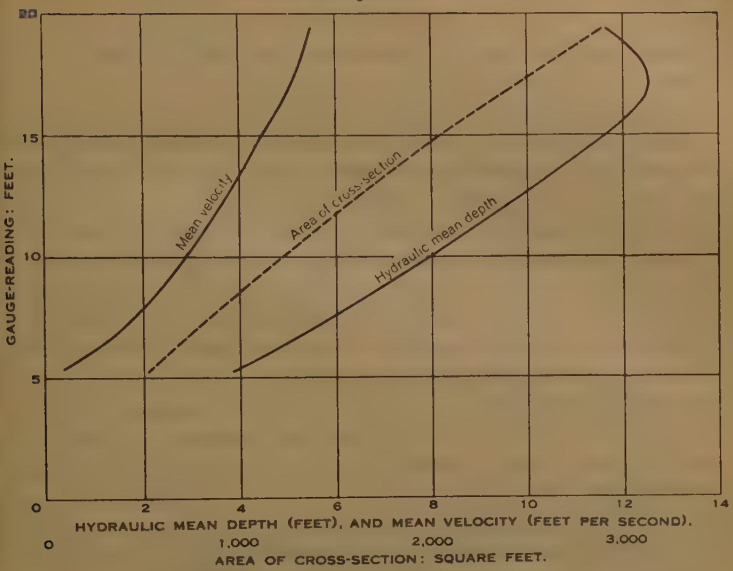
The Characteristics of the Gauging-Section.

The area, mean velocity and mean hydraulic radius of the section are plotted against the water-level scale-reading in *Fig. 4*. The lowest point at the bottom of the section is 1·4 foot below the zero of the scale, so that the maximum depth is greater than the scale-reading by this amount. It will be noticed that the curve for area is convex to the depth axis, indicating that the rate of change of area increases with increase in depth. The velocity curve, on the other hand, is concave to the depth axis, indicating a diminishing rate of increase of velocity with increase of depth. The curve for mean hydraulic radius (area divided by wetted perimeter) increases uniformly at first, but reaches a maximum of 12·55 feet at a scale reading of about 17·0 feet and then diminishes again. This is because the river has then begun to flow over the banks and there is a considerable width of shallow water on either side of the main channel, so that there is a greater increase in the wetted perimeter than in the area and the mean hydraulic radius is thus reduced.

The Stage–Discharge Curve.

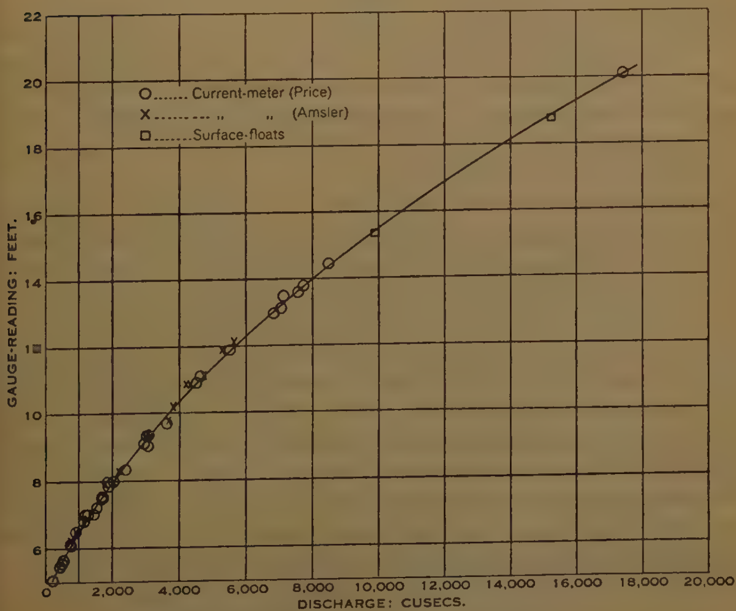
The results of fifty-nine discharge-measurements carried out during the years 1925–32 and one flood-discharge measured in 1922 have been plotted against the water-level reading on the gauging-

Fig. 4.



CHARACTERISTICS OF THE GAUGING-SECTION.

Fig. 5.



STAGE–DISCHARGE CURVE FOR GAUGING-SECTION.

section scale in *Fig. 5*. A mean curve has been drawn through the points so that the sum of the positive and negative errors of the individual points is zero. The method of gauging is indicated on the diagram.

The largest discharge, 17,400 cusecs, was measured at the Elan Aqueduct bridge on the 11th December, 1929, with the Price meter. Corrections were made for meter-drag, and it is considered that the results of the measurement can be relied upon as being within 5 per cent. of the true discharge. The other flood-measurements were made by means of surface-floats at the gauging-section. The value of these measurements is dependent on the choice of a float-coefficient.[1] The earlier, made in March, 1922, gave a discharge of 15,250 cusecs for a scale reading of 18·8 feet when a float-coefficient of 0·85 was used. Although it has since been found that the float-coefficient increases with the depth up to a scale reading of 14 feet, when the value is 0·93, the use of 0·85 for this gauging is held to be justified in view of the fact that the average depth across the section is actually less than when the river is flowing within its banks. The other float-measurement, made in March, 1933, gave a discharge of 9,900 cusecs for a scale-reading of 15·35 feet, using a float-coefficient of 0·93.

It is desirable to refer all discharge-measurements to a permanent scale at the actual gauging-section so as to avoid errors due to transference of the gauging to another scale some distance away. The stage–discharge curve for the gauging-section can be transferred to the other gauges by correlating the readings on the different gauges at various stages of the river. This enables Tables of discharges to be prepared giving the discharge in terms of the gauge-reading.

Before the scale was erected at the gauging-section in 1923 each discharge-measurement was referred to the Bewdley scale, readings being taken on this during the period covered by the measurement. The gaugings of 1921 and 1922 show a much greater individual scatter than those of subsequent years, and it was originally suggested that this might have been caused by the effects of weed-growth in the channel. Since 1923 no such variations in the stage–discharge relations have been observed, and it must be concluded that the weed-growth has no perceptible effect on the flow. Another possible cause of the discrepancies in the 1921 and 1922 measurements was the dumping of rubbish on the left bank of the river below Bewdley bridge and just above the rocky ledge which forms the control. The largest discrepancies occurred in the summer and autumn of 1922,

[1] This is discussed in Appendix I (p. 110).

gauging No. 12 made in December, 1922, giving a discharge of 23 per cent. less than that given by the curve.

The majority of the gaugings made in the spring and early summer of 1922 fell on the ordinary curve which agrees with the gaugings of subsequent years. On plotting the discharges against the readings on the Elan Aqueduct gauge the scatter is much less, and this indicates that the trouble must have been due to alterations in the control at Bewdley. It is also to be remembered that the gaugings in 1921 and 1922 were made before the observers had acquired much skill, and before the method of cable suspension for the meter had been adopted.

Altogether three different stage–discharge curves have been used in analysing the records :

(1) From March 1921 to July 1922, and from January 1923 to September 1923.
(2) From July to December 1922.
(3) From October 1923 to October 1936.

There was little difference between curves (1) and (3), showing that, apart from the happenings of 1922, the relation between stage and discharge at Bewdley was very consistent.

The Calculation of the Daily Discharges.

The daily discharges have been calculated from the 9-a.m. reading of the Bewdley scale by the aid of the Tables of discharges mentioned above. As a continuous record was available at Elan Aqueduct more frequent readings could have been obtained from the recorder-charts there, but it was found that a simple daily reading was sufficient, except very occasionally when the water-level was changing exceptionally rapidly.

The effect of more frequent readings was investigated for a period of three months, November 1931 to January 1932, during which the flow varied continuously and irregularly between wide limits. In November the discharge rose from 1,000 to 5,000 cusecs in a day and thereafter fluctuated between 8,000 and 3,000 cusecs. December was a normal winter month, the flow varying between 7,000 and 1,400 cusecs. Early in January there was very heavy rain, the flow increasing from 1,500 to 14,000 cusecs on the 9th and falling away again to 1,600 by the end of the month. Thus, if readings at more frequent intervals than once daily were required to give a true record of the discharge, the error of single daily readings should have been apparent during this period.

The charts from the automatic recorder were reduced in three ways :—

(1) The single daily readings at 9 a.m. were converted into flow in the usual way.

(2) The water-level was read off the chart at 6-hourly intervals (or 3-hourly intervals if the level were changing rapidly), and these four or eight readings were averaged to give an average water-level for the day, from which the flow was calculated.

(3) The four or eight readings for each day were each separately converted into flow and the average of the individual discharge taken as the flow for the day.

The daily discharges obtained by the three methods were then totalled and averaged in the usual way, and were as follows :—

Month.	Average daily discharge : cusecs.		
	(1)	(2)	(3)
November 1931	4,965	4,935	4,940
December 1931	2,655	2,700	2,700
January 1932	5,725	5,710	5,730
Average for the three months	4,445	4,445	4,456

Thus it is clear that for the Severn at Bewdley a single daily gauge-reading gives results which are indistinguishable from those obtained with four or eight readings per day. The maximum difference occurred in December 1931, when the flow as computed from single daily readings was 1·7 per cent. less than that obtained with more frequent readings. The relative slowness with which the discharge at Bewdley varies can be attributed to the size and shape of the catchment and to the effects of storage in the channel and in the pervious banks above Ironbridge.

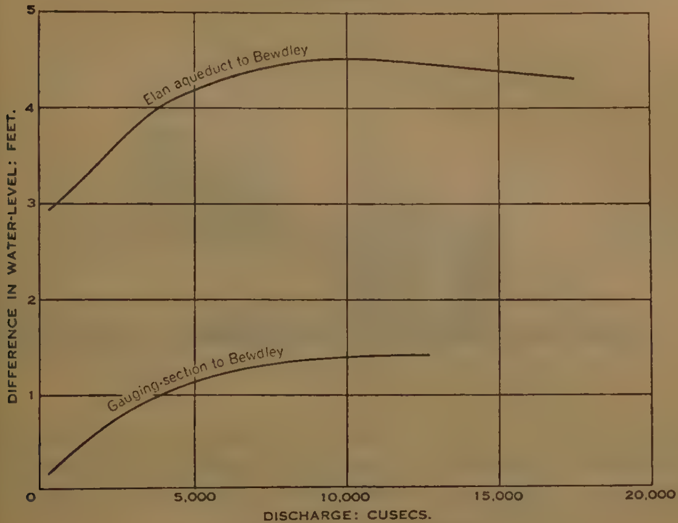
Comparison of the Different Gauges.

Since the channel between Bewdley and Elan Aqueduct bridge is free from artificial control the stage is everywhere a function of the discharge, provided the river is not rising or falling rapidly. By comparing the daily gauge-readings the relation between the gauges for all stages of flow can be determined. This relation then enables any doubtful observations to be checked and missing observations to be replaced.

The differences in level between Elan Aqueduct and Bewdley and between the gauging-section and Bewdley have been plotted in *Fig. 6* in terms of the discharge. As the channel is neither uniform nor straight, and the bed-slope is irregular, no attempt can be made

to compare the observed relations between surface-slope and discharge with any of the formulas for calculating the discharge of open channels. It will be noted that the difference between Elan

Fig. 6.



CORRELATION OF GAUGES.

Aqueduct and Bewdley gauges reaches a maximum of 4·5 feet when the discharge is about 10,000 cusecs—the " bank full " stage above Bewdley.

DESCRIPTION OF TYPICAL HYDROGRAPHS.

The daily discharges have been plotted to form hydrographs for each of the 15 years covered by records. The hydrographs are related to a " water year " commencing on 1st October because this period is generally considered to be more suitable than the calendar year for use in comparing the relations between rainfall and run-off. Five hydrographs have been selected for reproduction,[1] each of which illustrates some special features.

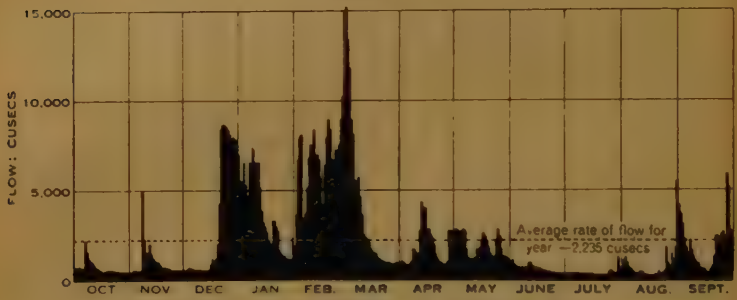
1922–23 (*Fig. 7*, p. 96) represents an average year, the average rate of flow most nearly approaching the average for the whole period. The river rose in every month, but in June, July, August and October the rises were insufficient to bring the flow up to the average

[1] The ten hydrographs not reproduced will be placed with the MS. of the Paper in the Institution Library.—SEC. INST. C.E.

for the year. The biggest discharges occurred in the course of two wet periods in December and January, and in February and March. The maximum discharge was 15,250 cusecs on the 2nd March and the minimum 300 cusecs on the 17th August.

1927–28 (*Fig. 8*) had the second highest winter rainfall of the series,

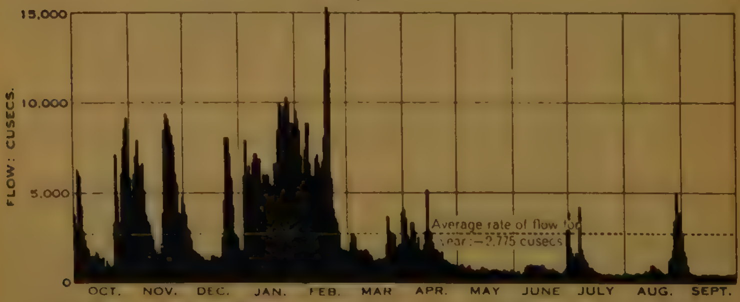
Fig. 7.



HYDROGRAPH FOR 1922–23, AN AVERAGE YEAR.

numerous rises occurring in October, November and December, with 2 months' high water in January and February. The maximum discharge was 16,100 cusecs on the 18th February and the minimum 440 cusecs in July, August, and September.

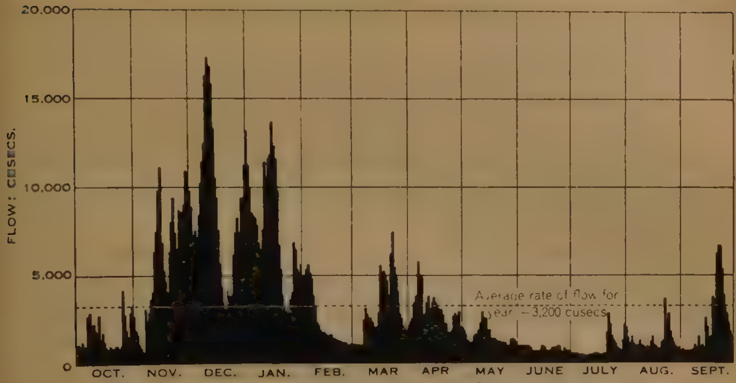
Fig. 8.



HYDROGRAPH FOR 1927–28, A YEAR WITH A HIGH WINTER RAINFALL.

1929–30 (*Fig. 9*) was the wettest year and gave the largest discharge of the series. The run-off was mainly concentrated in the months of November, December, and January, but minor rises occurred in the other months. The maximum discharge, 17,400 cusecs, occurred on the 11th December, the flow was in excess of 10,000 cusecs for 22 days in all, and the minimum discharge was 260 cusecs in October.

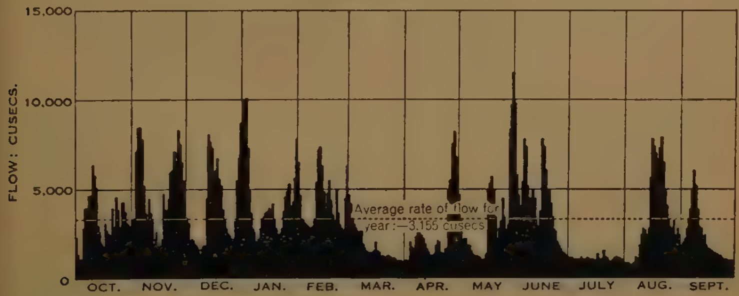
Fig. 9.



HYDROGRAPH FOR 1929–30, THE WETTEST YEAR.

1930–31 (*Fig. 10*) had a particularly wet summer, and the hydrograph shows very frequent but short rises all through the year.

Fig. 10.

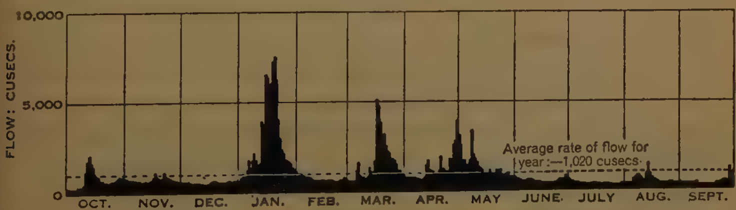


HYDROGRAPH FOR 1930–31, A YEAR WITH A HIGH SUMMER RAINFALL.

The maximum discharge was 11,600 cusecs on the 31st May and the minimum 760 cusecs in August.

1933–34 (*Fig. 11*) was the driest year of the series, and the discharge was less than one-third of that of the wettest year. The

Fig. 11.



HYDROGRAPH FOR 1933–34, THE DRIEST YEAR.

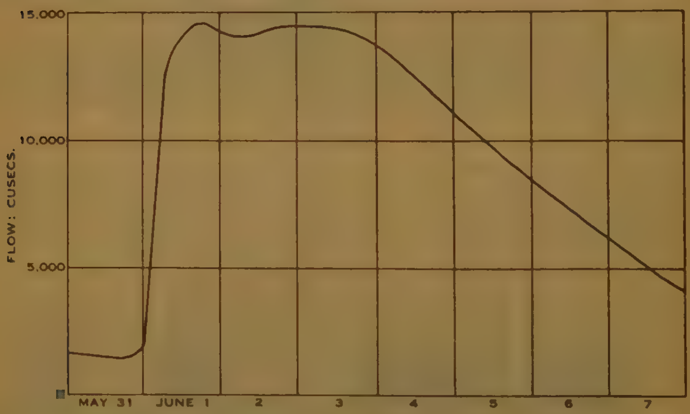
only rises of any moment were in January and March. The maximum discharge was 7,500 cusecs on the 20th January, and the minimum 260 cusecs in October.

Floods.

The hydrographs of three floods taken from the recording gauge at Elan Aqueduct bridge have been selected for description on account of special features.

31 May–1 June, 1924 (*Fig. 12*).—On these two days there was widespread heavy rain over Shropshire and Worcestershire, and severe floods were produced at many places. The rain commenced on the

Fig. 12.



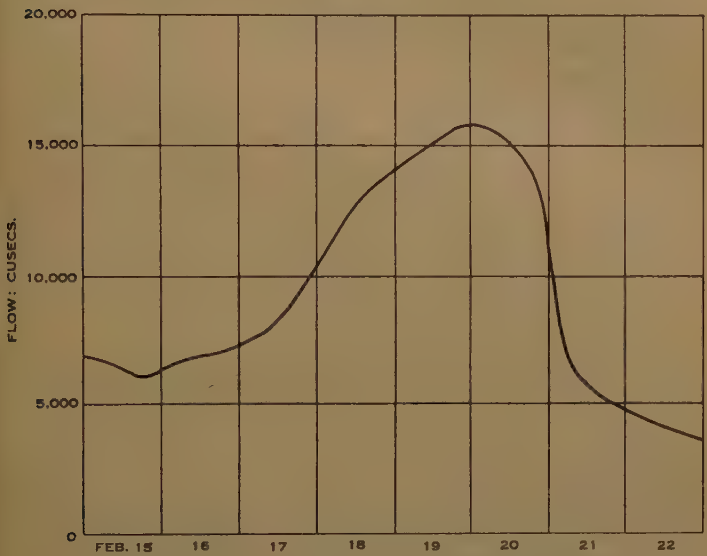
Hydrograph of Flood of 31 May–1 June 1924.

evening of the 31st May and continued throughout the night. The total rainfall over the Severn valley was between 3 and 4 inches. The hydrograph shows that the river began to rise about 6 p.m. on the 31st May, but after midnight the rise became very rapid and the discharge increased from 1,600 cusecs to 11,000 cusecs at 6 a.m., reaching a maximum of 14,600 cusecs at 6 p.m. on the 1st June. The discharge remained at more than 14,000 cusecs for 2½ days and then diminished very gradually, although there was only about 0·4 inch of rain on the 1st June and the following days were fine. The duration of the 14,000-cusec discharge must be attributed to the gradual run-off of the heavy rainfall. Had 3½ inches been discharged in 1 day the peak of the flood would have been 180,000 cusecs, or fourteen times the actual peak. Of this rainfall it is estimated that only about 48 per cent. had been discharged by the

time the river discharge had returned to normal. Some of the remaining 52 per cent. may have gone into ground storage, but most of it was probably lost.

16–22 February, 1928 (*Fig. 13*).—This was an ordinary winter flood which rose gradually to a peak discharge of 16,100 cusecs on

Fig. 13.



HYDROGRAPH OF FLOOD OF 16–22 FEBRUARY 1928.

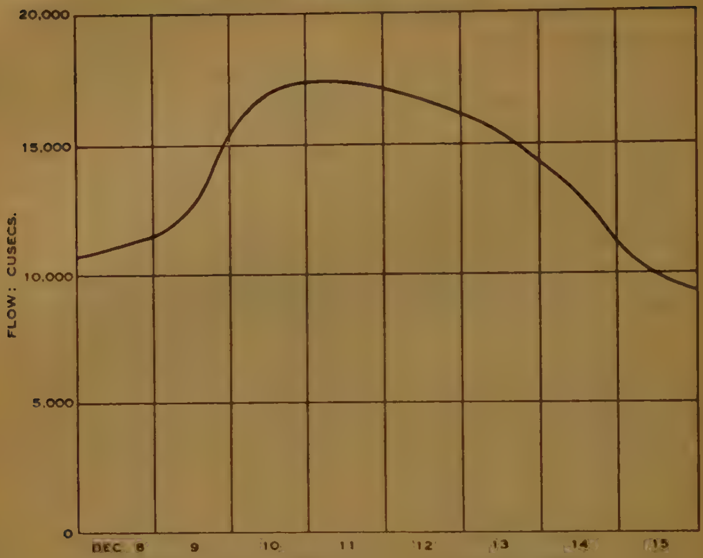
the 18th. The record is remarkable for the exceptionally sudden drop in discharge which occurred on the night of the 20th–21st February.

11 December, 1929 (*Fig. 14*, p. 100).—The discharge on this day, 17,400 cusecs, was very close to the highest recorded during the period of observation. It will be noted that the river rose gradually in consequence of repeated heavy rains, and after the maximum had been passed the discharge fell off very gradually. A current-meter discharge-measurement was made at the peak of this flood.

DURATION-CURVES.

Curves showing the proportion of the year during which the flow was equal to any given value are plotted in *Fig. 15* (p. 100). The plotted curves refer to (1) 1933–34, the year of lowest flow; (2) 1929–30, the year of highest flow; (3) 1930–31, with a very wet summer; and (4) the average for the whole period 1921–36. The duration-

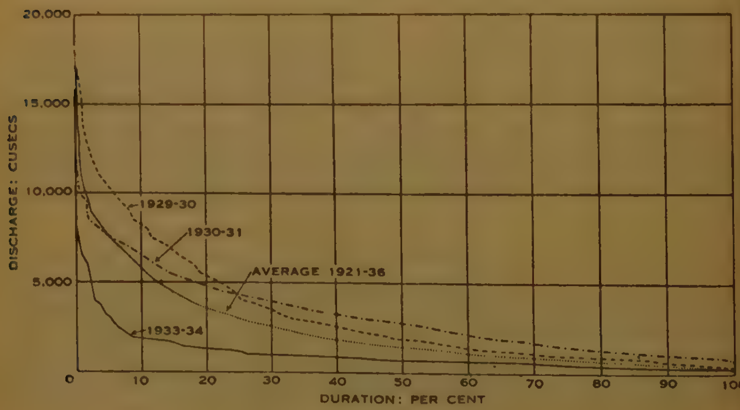
Fig. 14.



HYDROGRAPH OF FLOOD OF 11 DECEMBER 1929.

curves bring out, perhaps even more clearly than the hydrographs, the great variations between the individual years. There was only a slight difference between the average flows of the two years 1929–30 and 1930–31, but in the former year the discharge occurred mainly concentrated in a few large floods, whereas in the latter year the rainfall and discharge were more uniformly distributed and the

Fig. 15.



DURATION-CURVES.

flow remained fairly high throughout the year; the loss, however, was greater.

The Variations in Discharge.

The hydrographs have been plotted with the discharge expressed in cusecs, but for a more general discussion of the results it is convenient to convert these quantities into inches of depth on the catchment. In this way the information has been put into a form which is directly available for comparison with the results obtained on other rivers. The monthly average discharges are given in Table I (p. 102), together with the yearly totals, and the monthly and yearly averages for the 15 years 1921–36.

The average quantity discharged per annum was 18·90 inches, the maximum year's discharge was 26·63 inches or 141 per cent. of this, and the minimum 8·49 inches or 45 per cent. The average annual discharge for the three wettest consecutive years (1929–32) was 24·19 inches and for the three driest years (1932–35) 12·69 inches. The maximum and minimum monthly variations expressed as percentages of the monthly averages are also shown in the Table.

The lowest month was July 1921, when the total run-off was only 0·15 inch; this was closely followed by September 1933 with 0·18 inch. It will be remarked that in each case the month of minimum run-off is the fourth of a series having small and gradually diminishing discharges. The highest month's discharge was 6·21 inches in December 1929, 41 times that of the lowest month.

The lowest discharge was 200 cusecs in July 1921, and the highest flood-discharge 18,200 cusecs in January 1925, 91 times the minimum.

The Rainfall.

The average distribution of annual rainfall over the catchment is shown on *Fig. 1* (p. 83), which is based on the period 1881–1915 taken as standard by the British Rainfall Organization. Records from about a hundred gauges in and adjoining the area were utilized. As only a few of these gauges were in existence throughout the whole period the averages for records which did not cover the full period were adjusted in accordance with the results obtained at adjoining long-period gauges. The average rainfalls at the different stations were plotted on a ½-inch scale map and isohyetal lines (lines of equal rainfall) were drawn, making use of the general principles laid down by Dr. H. R. Mill and the late Mr. Carle Salter.[1] The

[1] See, for example, J. Glasspoole, "The Rainfall of Norfolk." *British Rainfall*, 1928, p. 270.

TABLE I.

RUN-OFF (inches).

	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Total for year.	Percentage of 15-year average.
1920–21							0·69 *	0·45 *	0·27 *	0·15 *	0·44 *	0·23 *		
21–22	0·42	0·82	1·29	3·49	2·92	3·16	1·32	0·75	0·26	0·51	0·41	0·70	16·05	85
22–23	0·50	0·67	2·30	2·72	4·24	2·63	1·29	1·40	0·45	0·30	0·64	1·48	18·62	99
23–24	2·80	2·68	1·97	3·07	0·78	0·89	0·77	2·01	2·63	1·03	1·51	1·97	22·11	117
24–25	2·48	2·23	2·83	3·78	3·81	1·66	0·96	0·93	0·51	0·27	0·39	0·55	20·40	108
25–26	1·34	1·53	1·69	3·91	2·36	1·16	0·48	0·96	0·43	0·55	0·63	0·62	15·66	83
26–27	1·36	3·97	0·90	1·89	1·58	2·16	1·98	0·71	0·81	1·15	2·31	1·87	20·69	110
27–28	2·39	3·55	1·98	4·90	4·37	1·40	1·46	0·55	0·59	0·75	0·79	0·44	23·17	122
28–29	1·52	4·14	1·97	1·43	1·41	0·63	0·39	0·57	0·42	0·29	0·59	0·29	13·65	72
29–30	1·10	3·76	6·21	5·36	1·50	2·09	1·90	1·08	0·57	0·61	0·93	1·52	26·63	141
30–31	2·22	3·31	2·75	3·02	2·63	1·26	1·67	2·16	2·84	0·79	2·17	1·43	26·25	139
31–32	0·64	3·34	1·83	3·98	0·68	0·80	2·62	2·35	0·81	1·11	0·70	0·82	19·68	104
32–33	2·12	1·86	1·51	1·97	2·38	2·90	0·60	0·52	0·58	0·47	0·23	0·18	15·32	81
33–34	0·56	0·54	0·43	2·07	0·50	1·18	0·88	0·99	0·39	0·26	0·39	0·30	8·49	45
34–35	1·04	0·90	2·49	1·60	2·50	1·39	1·58	0·57	0·65	0·44	0·28	0·82	14·26	75
35–36	2·14	3·57	2·56	3·42	2·12	1·75	1·53	0·69	1·14	2·10	0·64	0·90	22·56	119

Monthly averages for the 15-year period October 1921 to September 1936:—

	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.
	1·51	2·46	2·18	3·11	2·25	1·67	1·29	1·08	0·87	0·71	0·84	0·93

Maximum and minimum percentages of the monthly averages:—

	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.
Max.	195	168	285	172	194	173	201	218	327	296	275	212
Min.	28	22	20	46	22	38	30	42	30	21	27	19

Average run-off for the 15-year period October 1921 to September 1936 . . . . 18·90 inches

NOTE.—* Indicates figures not included in the averages.

configuration of the land in relation to the prevailing wind is the controlling factor in long-period rainfall-distribution, and it will be seen that the highest rainfall occurs on the mountains on the western boundary of the catchment. The rainfall exceeds 80 inches over a small area mainly above the 1,500-foot contour on the slopes of Plynlimmon, on the south-western corner of the map. Areas with more than 70 inches occur there and at the head of lake Vyrnwy. The 60-inch line is parallel to the 70-inch, but covers a larger area, including some land below the 1,000-foot contour. The shape of the 50-, 40- and 35-inch lines follows the configuration of the ground, the lines running up the valleys. Two small isolated areas with more than 35 inches occur on Breidden hill, and on the Long Mountain. The 30-inch isohyet bends around these areas, and patches with more than 30 inches occur in the south near Church Stretton and near Cleobury. Areas with more than 27·5 inches occur along the eastern boundary, and an elongated area with less than 25 inches to the east of Shrewsbury.

The average rainfall for the period 1881–1915 was 34·6 inches.

The monthly rainfall on the catchment has been derived from manuscript maps prepared by the British Rainfall Organization (Meteorological Office). These maps are on a scale of 1 inch to 19 miles, and it is not suggested that a high degree of precision can be obtained in measuring the rainfall over the relatively small area covered by the Severn catchment. It is considered, however, that the advantages of the cartographic method of discussing rainfall-distribution outweigh the disadvantage of the small scale of the maps, and that measurements on these maps give more accurate results than could have been obtained by any other method with the resources available. A check on the accuracy of the measurement of the rainfall-areas is afforded by comparing the total of the 12 individual months with the quantity measured on the map showing the total year's rainfall. The maximum discrepancy between the two totals was about 3 per cent., and the difference was usually much less. The monthly rainfalls are given in Table II (p. 104), together with the annual totals, the monthly and annual averages and the percentage-variations, as in the case of the river-discharge.

During the 15 years 1921–36 March had the lowest average rainfall, and October the highest. The lowest month's rainfall was 0·35 inch in February 1934, and the highest was 8·46 in November 1929. The average annual rainfall of the period, 38·14 inches, was 103 per cent. of the long-period average (1881–1915).

The rainfall of the wettest year, 1929–30, was 147 per cent. of the long-period average, and that of the driest year, 1933–34, was 75 per cent. of it.

TABLE II.

RAINFALL (inches).

	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Total for year.	Percentage of 15-year average.	Percentage of a long average.
1920–21							1·41*	1·61 *	0·41 *	1·08 *	4·18 *	0·95 *			
21–22	2·48	2·89	3·72	4·81	4·40	3·12	2·90	1·16	1·28	4·28	3·08	2·68	36·80	96	107
22–23	1·03	1·56	4·72	2·88	6·88	1·84	2·66	3·20	0·64	3·43	4·00	2·97	35·81	94	104
23–24	5·85	2·98	3·68	3·08	0·66	1·65	3·34	5·67	2·91	5·20	4·26	4·37	43·65	114	126
24–25	5·04	2·56	4·75	3·47	5·18	1·32	2·48	3·72	0·92	2·95	3·61	3·92	39·92	105	115
25–26	4·67	2·44	3·88	4·21	2·81	1·48	2·03	3·69	1·81	3·72	3·22	2·29	36·25	95	105
26–27	3·68	6·48	0·99	4·10	2·71	3·64	2·55	1·78	5·06	3·38	6·22	4·37	44·96	118	130
27–28	3·45	4·59	2·34	6·41	3·60	2·48	1·86	1·20	4·38	1·88	4·16	1·62	37·97	100	110
28–29	5·18	6·01	2·92	1·80	0·83	0·46	0·95	2·32	2·67	2·45	2·63	1·48	29·70	78	86
29–30	4·80	8·46	8·07	6·05	0·72	3·32	3·32	2·45	1·35	4·32	3·43	4·55	50·84	133	147
30–31	4·85	4·71	4·56	2·69	3·03	0·55	4·12	5·28	4·53	3·86	5·70	2·95	46·83	123	136
31–32	1·13	5·98	1·96	3·95	1·45	2·98	4·76	4·68	0·93	5·36	1·86	3·19	38·23	100	111
32–33	5·68	2·54	2·01	2·44	4·07	2·57	0·93	1·30	2·86	2·43	1·18	0·94	28·95	76	84
33–34	4·87	1·16	0·69	3·53	0·35	2·68	2·86	1·84	1·57	1·55	2·80	2·16	26·06	68	75
34–35	3·38	2·20	5·62	1·48	4·43	1·03	3·86	1·05	3·38	0·79	1·60	5·30	34·12	89	99
35–36	5·57	5·02	3·66	4·60	2·34	2·70	2·22	1·56	3·79	5·43	1·26	3·86	42·01	110	121

Monthly averages for the 15-year period 1921–36 :—

	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.
	4·11	3·97	3·57	3·70	2·90	2·12	2·72	2·73	2·54	3·40	3·27	3·11

Maximum and minimum percentages of the monthly averages :—

	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.
Max.	142	213	226	173	237	17	175	208	199	160	190	170
Min.	25	29	19	40	12	22	34	38	16	23	36	30

Average rainfall for the 15-year period October 1921 to September 1936 . . . . 38·14 inches

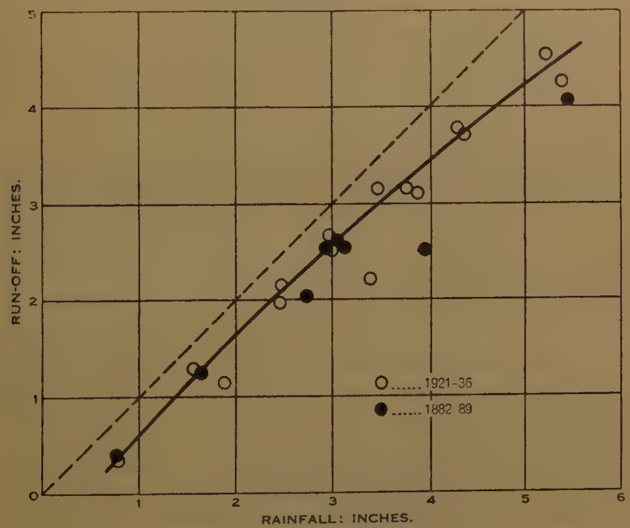
Long average rainfall for the 35-year period 1881–1915 . . . . 34·6 inches

NOTE.—* Indicates figures not included in the averages.

The Relation between Rainfall and Run-off.

The difference between the average annual rainfall and run-off (for the year commencing October 1st) is 19·24 inches, or slightly more than half the rainfall. The largest loss, 23·80 inches, was in the year 1926–27, which had a high rainfall fairly evenly distributed throughout the year. The smallest loss was 13·63 inches in 1932–33, a year of low rainfall. The monthly and annual losses are shown in Table III (p. 106). During the months December to March the run-off exceeded the rainfall on several occasions, and the loss for the month was negative. This was due to the discharge of rain that

Fig. 16.



Rainfall and Run-Off in July.

had fallen in the previous month. The existence of this condition of augmented flow renders impossible any study of the relation between the rainfall and run-off during the winter based on monthly periods. In summer there is a much higher rate of loss, and little of the rain appears as discharge. If the rainfall and run-off are plotted for the months June, July and August it is found that there is a definite relation between them, as shown in *Fig. 16*, which relates to the rainfall of July.

At the beginning of this investigation the question of the most suitable date for starting the year was considered. October 1st was selected as representing the end of summer conditions, and it

TABLE III.
Loss (Inches).

	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Total for year.	Percentage of 15-year average.
1920–21							0·72 *	0·16 *	0·14 *	0·93 *	3·74 *	0·72 *		
21–22	2·06	2·07	2·43	1·32	1·48	*0·04*	1·58	0·41	1·02	3·77	2·67	1·98	20·75	108
22–23	0·53	0·89	2·42	0·16	2·64	*0·79*	1·37	1·80	0·19	3·13	3·36	1·49	17·19	89
23–24	3·05	0·30	1·71	0·01	*0·12*	0·76	2·57	3·66	0·28	4·17	2·75	2·40	21·54	112
24–25	2·56	0·33	1·92	*0·31*	1·37	*0·34*	1·52	2·79	0·41	2·68	3·22	3·37	19·52	101
25–26	3·33	0·91	2·19	0·30	0·45	0·32	1·55	2·73	1·38	3·17	2·59	1·67	20·59	107
26–27	2·32	2·51	0·09	2·21	1·13	1·48	0·57	1·07	4·25	2·23	3·91	2·50	24·27	126
27–28	1·06	1·04	0·36	1·51	*0·77*	1·08	0·40	0·65	3·79	1·13	3·37	1·18	14·80	77
28–29	3·66	1·87	0·95	0·37	*0·58*	*0·17*	0·56	1·75	2·25	2·16	2·04	1·19	16·05	83
29–30	3·70	4·70	1·86	0·69	*0·78*	1·23	1·42	1·37	0·78	3·71	2·50	3·03	24·21	126
30–31	2·63	1·40	1·81	*0·33*	0·40	*0·71*	2·45	3·12	1·69	3·07	3·53	1·52	20·58	107
31–32	0·49	2·64	0·13	*0·03*	0·77	2·18	2·14	2·33	0·12	4·25	1·16	2·37	18·55	96
32–33	3·56	0·68	0·50	0·47	1·69	*0·33*	0·33	0·78	2·28	1·96	0·95	0·76	13·63	71
33–34	4·31	0·62	0·26	1·46	*0·15*	1·50	1·98	0·85	1·18	1·29	2·41	1·86	17·57	91
34–35	2·34	1·30	3·13	*0·12*	1·93	*0·36*	2·28	0·48	2·73	0·35	1·32	4·48	19·86	103
35–36	3·43	1·45	1·10	1·18	0·22	0·95	0·69	0·87	2·65	3·33	0·62	2·96	19·45	101

Average monthly loss for the 15-year period October 1921 to September 1936 :—
2·60 1·52 1·39 0·59 0·65 0·45 1·43 1·64 1·67 2·69 2·43 2·18

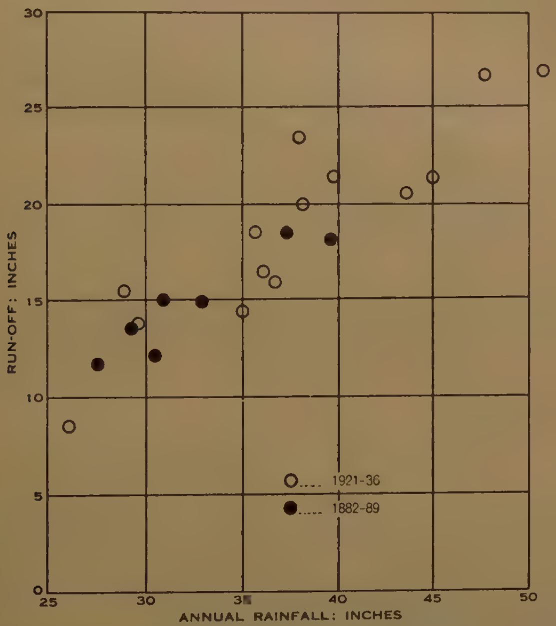
Average annual loss for the 15-year period October 1921 to September 1936 . . . . 19·24 inches

NOTE.—* Indicates figures not included in the averages.

Negative losses are shown in italics.

was thought that the quantity of water retained in ground storage would be a minimum at about that date. In *Fig. 17* the annual run-off has been plotted against the rainfall for the 15 years 1921–36. The records for the years 1882–89, as calculated by Mr. J. S. Owens, M.D., Assoc. M. Inst. C.E.,[1] have also been plotted on this figure, but it must be noted that they were obtained on a catchment of 1970 square miles area—21 per cent. greater than the catchment

Fig. 17.



Annual Rainfall and Run-Off; Year Commencing 1 October.

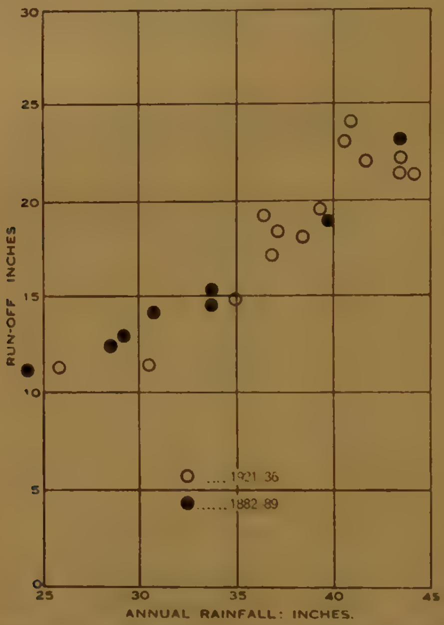
to Bewdley. The scatter of the individual points is considerable, and the 1882–89 points fall within the more recent observations.

When the run-off and rainfall are plotted for the calendar year, commencing January 1st, similar results are obtained (*Fig. 18*, p. 108). It should be noted that Dr. Owens found that a simple relationship existed between rainfall and loss for the 1882–89 results when worked out for the calendar year. The 1921–36 results do not confirm this conclusion, although some of the individual points fall on or near the 1882–89 curve. From a general consideration of the factors

[1] The Investigation of Rivers, Final Report, p. 9. Royal Geographical Society, 1916.

which control the loss on a catchment it is difficult to see why there should be a simple relation between rainfall and run-off, particularly when the calendar year is taken. Starting the year on the 1st January

Fig. 18.



ANNUAL RAINFALL AND RUN-OFF; YEAR COMMENCING 1 JANUARY.

when the ground storage is very full means that a considerable proportion of the previous year's rainfall will flow off in the early months, and if the rainfall in these months be low, a condition of augmented flow—in which the run-off exceeds the rainfall—may obtain.

ACKNOWLEDGEMENTS.

This research was carried out from the Civil Engineering Department of the Imperial College of Science and Technology, and the Authors are indebted to the College Authorities for the facilities afforded, and to many students for their assistance in the field work. Part of the work was done while one of the Authors (Dr. Hogan) was Technical Officer to the Committee on Gauging Rivers and Tidal Currents of the Department of Scientific and Industrial Research, and the Authors are grateful to Sir Henry Lyons, F.R.S., Chairman of

that Committee, for his continued interest in the research. The continuation of the work since 1923 was assisted by the grants from the D.S.I.R. in the aid of travelling expenses.

The daily gauge-readings at Bewdley were taken by the boys of the Church of England School, and the Authors are greatly indebted to Mr. J. Bates, under whose supervision the records of the gauge-readings have been kept. The daily gauge-readings and the automatic-gauge charts at Elan Aqueduct bridge have been kept by the staff of the Birmingham Corporation Water Department. The establishment of the gauge and recorder was rendered possible by the active interest shown by the late Mr. F. W. Macaulay, M. Inst. C.E., in the research, and thanks are due to Mr. J. W. Wilkinson, M. Inst. C.E., the present Chief Engineer, for continuing the records.

In studying the rainfall of the district the Authors have been greatly assisted by the British Rainfall Organization, Meteorological Office, and they wish to thank Dr. John Glasspoole for his advice on many points arising out of the work.

The Paper is accompanied by thirty sheets of drawings, from some of which the Figures in the text have been prepared, and by the following Appendixes.

APPENDIX I.

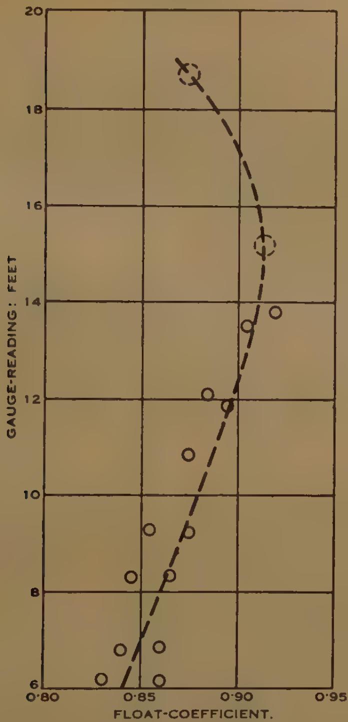
Discharge-Measurements by Surface Floats.

Surface-float observations were made simultaneously with the current-meter discharge-measurements when the help of a class of students was available. The floats were timed over a distance of 200 feet, 100 feet above and 100 feet below the gauging-section. The position of each float at entry and exit from the measured length was fixed by a theodolite, and the location of the observation on the gauging-section was determined graphically from these positions. Efforts were made to obtain as uniform as possible a distribution of the observations across the section, and from 30 to 60 floats were used for each gauging. The velocities of the different floats were plotted and averaged graphically, and the velocities at 10-foot intervals read off the curve. Since the average velocity is less than the surface-velocity, the float-velocities require to be reduced by the application of a factor less than unity known as the "float-coefficient" to enable the discharge to be calculated. Actually, in these combined measurements the discharge had been obtained by the current-meter and so the "float-coefficient" was determined. This was done by calculating a false discharge from the measured depths and surface-velocities and dividing that into the true discharge. The operation determined an average float-coefficient for the whole cross-section. Owing to the shape of the vertical velocity-curves it would be expected that the coefficient would be higher for the verticals of greater depth and velocity and less for those in the shallower and slower parts of the section, but it is suggested that the use of float-coefficients which differed at different parts of the section would be an unnecessary complication. On the other hand, there is definite evidence that the average float-coefficient varies with the depth at the gauging-section and the discharge, as shown in *Fig. 19*. In considering this diagram it must be noted that the depth at the middle of the section does not fall below about 6 feet even at the lowest flow, and rises to about 15 feet at the highest flow at which comparisons were made, the corresponding average velocities ranging from 0·75 to 4·6 feet per second. Similar variations in the value of the float-coefficient with depth and discharge are to be anticipated in other rivers, but the actual magnitude of the coefficient will vary with the relation between depth and velocity and with the roughness of the channel.

There is evidence that reduction occurs in the float-coefficient when the discharge exceeds 10,000 cusecs and flooding commences, as indicated by the dotted points on *Fig. 19*.

The average scatter of the individual points on either side of the curve ranges from 3 per cent. at the lower flows to 1·5 per cent. at the higher, showing that, provided the true value of the coefficient is known, surface-float gaugings are capable of giving results within 4 per cent. of the true discharge (assuming that the depths are accurately measured). If, however, an average of 0·85 had been assumed the maximum error of an individual gauging would have been about 11 per cent.

Fig. 19.



VARIATIONS OF FLOAT-COEFFICIENT WITH DEPTH.

APPENDIX II

CURRENT-METER RATING.

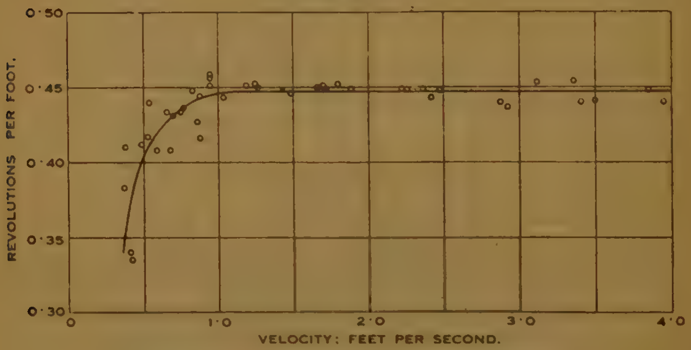
The calibration of the meters used on the Severn was done in the rating channel at the Imperial College. The meters were calibrated before being put into use, and the calibration was checked at least once a year afterwards. The channel, 4 feet 6 inches wide, was filled to a depth of 3 feet and the meter was run 1 foot below the surface. The trolley from which the meter is suspended runs on rails on either side of the channel and was propelled by hand. It was found that a practised operator could keep the speed constant to within 5 per cent. over the test length of about 60 feet.

Various methods of timing the speed of the carriage and revolutions of the

meter are employed in different laboratories, but the method used on this work has the advantage of simplicity because it does not involve the use of electrical recording. Two stop-watches were used, one to measure the velocity of the trolley over a given distance, and the other to time the revolutions of the meter whilst the carriage was running at constant velocity. Contact studs 5 feet apart were arranged along the channel and caused a lamp to flash as the carriage passed; by timing the interval between the flashes the velocity could be determined. Another lamp was connected in circuit with the meter so that the revolutions of the meter were also indicated and could be timed from lamp flashes. Each run gives the number of revolutions per second of the meter when turned at a given velocity, and by plotting a number of such points it is possible to draw a calibration curve.

In practice it is found more satisfactory to plot the number of revolutions per foot of travel (the number of revolutions per second divided by the velocity) against the velocity, because this type of diagram is better suited to detecting errors in the observations, or irregularities in the behaviour of the meter. A

Fig. 20.



CALIBRATION-CURVE OF PRICE-TYPE METER.

typical series of results for a Price-type meter is shown in *Fig. 20.* It will be seen that the revolutions per foot increase rapidly at first and finally become constant at a velocity of about 1 foot per second. At higher speeds the relation remains constant (except for accidental effects due to wave-making resistance in the channel). A complete calibration consisted of between fifty and sixty observations at velocities ranging from about ¼ to 6 feet per second. This large number of observations was required in order to enable a fair curve to be drawn owing to the scatter of the individual points. The scatter was due to variations in the rotation and signals of the instrument caused by variations in the frictional resistance and in the behaviour of the electrical contacts during the different observations. The condition of the bearings of the meter has an important influence on the part of the calibration curve below about 2 feet per second and care should be taken to see that they are maintained in good condition.

When the curve connecting revolutions per foot and velocity has been drawn it may be used to prepare a calibration Table giving the velocity corresponding to various measured time-intervals for a given number of revolutions of the meter, as shown in Table IV.

TABLE IV.—CURRENT-METER RATING.

Time for given number of revolutions: seconds.	Velocity: feet per second.							
	5	10	15	20	30	50	80	100 revs.
40	0·34	0·60	0·86	1·13	1·69	2·81	4·50	5·62
45	0·32	0·54	0·77	1·01	1·50	2·50	4·00	5·00
50	0·30	0·49	0·70	0·91	1·35	2·25	3·60	4·50
55	0·29	0·46	0·65	0·84	1·23	2·05	3·27	4·09
60	0·28	0·42	0·60	0·77	1·13	1·88	3·00	3·75
65	0·27	0·40	0·56	0·72	1·05	1·73	2·77	3·46
70	0·26	0·38	0·52	0·67	0·97	1·61	2·57	3·21
75	0·25	0·36	0·49	0·63	0·91	1·51	2·39	2·99
80	0·24	0·34	0·47	0·60	0·85	1·41	2·24	2·81

NOTE.—In actual use the Table would contain entries for each second of time interval from 40 to 80 seconds.

Discussion.

Dr. Hogan.

Dr. HOGAN observed that the largest flood measured during the course of the work was 18,200 cusecs (p. 101) which was a surprisingly low figure, but in the Report of the Floods Committee [1] a figure was given of 40,000 cusecs, which had been measured in the year 1886 on a slightly larger catchment-area than Bewdley. The run-off for the 1886 flood had been 19·6 cusecs per square mile, which was 76 per cent. greater than the maximum of the present series of records.

Dr. Hogan then showed a number of lantern-slides illustrating the work described in the Paper.

Mr. Binnie.

Mr. W. J. E. BINNIE said that the published data with regard to river-flow in Great Britain were so scanty that the Paper would prove of very great value. Great Britain was almost unique amongst civilized countries in the apathy which had been shown by the Government in the past with regard to the collection of suitable statistics. The Land Drainage Act of 1930 had caused certain Catchment Boards to be set up and had made provision whereby further Boards could be set up. Those Boards were endowed with ample borrowing powers and they could undertake the systematic collection of statistics, and it was obvious that, arrangements having been made for obtaining all those statistics, some central authority should be set up whose duty it would be to analyse and publish them. The Government had recently appointed the Inland Water Survey Committee with that object in view. Water engineers had attempted in the past to collect information on the subject, and, whilst a good many authorities were quite willing to supply it, there were others who were very reluctant to disclose the figures in case they might be used against them at some future inquiry. He thought that it would be preferable that, although there was no doubt that a large number of statistics could be voluntarily collected, it should be made obligatory to furnish them. He understood that the Inland Water Survey Committee had no financial powers and that any expenditure incurred was met by the Ministry of Health. In paragraphs 29 and 30 of their Report last year, that Committee set out their intention with regard to the publishing of statistics. They themselves apparently proposed only to publish an annual report in which levels and occasional gaugings would be given, and that the Ministry of

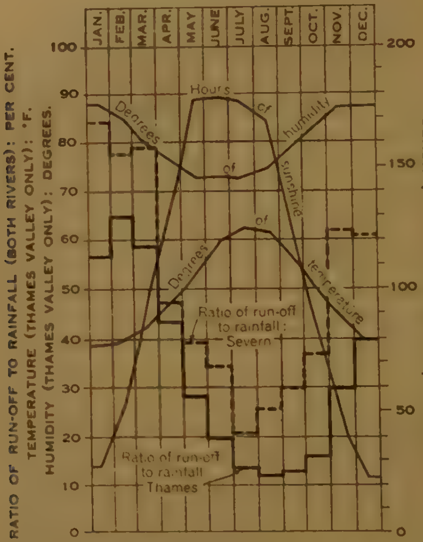
[1] "Interim Report of the Committee on Floods in Relation to Reservoir Practice." Inst. C.E. 1933.

alth should be entrusted with the task of issuing about once a ır Tables of actual volumetric measurements, which was the ormation engineers required. Mr. Binnie.

Dr. Hogan had referred to the fact that during the period of ıging there had been no large flood on the river. It appeared that lood of 17,400 cusecs had occurred in 1929, but that only repre-ted about 17 cusecs per thousand acres of catchment-area. The hest discharge was given (p. 101) as 18,200 cusecs, and floods ching 15,250 cusecs in 1922, 14,600 cusecs in 1924 and 16,100 ecs in 1928 were also recorded. Those figures gave the impression t they referred to a flood which occurred every few years, and , say, once in 100 years, for which provision should be made in ervoir-construction. The lowest dry-weather flow—200 cusecs— s very low for a catchment-area of the geological character and ent of that dealt with in the Paper, but he thought that the lanation was given on p. 84, where the Authors pointed out that it s probable that a considerable quantity of water might be lost to river by percolation along the direction of its old channel, par-ılarly at low stages of flow. The monthly run-off and rainfall en in Table I (p. 102), and in Table II (p. 104), were extremely ful; the average annual rainfall for the period from 1921 to 1936 s 38·14 inches while the run-off was 18·9 inches, giving a difference 19·24 inches, the proportion of annual rainfall to run-off being proximately 50 per cent. The difference between rainfall and -off depended not only on the amount of the rainfall, but also on perature, prevalence of sunshine, humidity, the geological racter of the drainage-area, the character of the vegetation, the epness of the valley-slopes, and other factors. It was therefore extremely complicated problem to deduce what the actual run-off ıld be from figures of rainfall, because those statistics were not ilable. They were, however, available in the case of the Thames everal observatories were situated in the area, records of tempera-e, hours of sunshine, and humidity being available for a period of ny years. The effect of those factors was shown in *Fig. 21* (p. 116), ich gave the monthly variation in the ratio of the run-off to the ıfall for both the Thames and the Severn, and also the changes of perature and humidity, as well as the hours of sunshine. The res of temperature and humidity, and the hours of sunshine, were the Thames only, as there were no observations available for the ern. The effect of sunshine, temperature and humidity on the ratio the run-off to the rainfall during any month was clearly shown. n the past, in the absence of actual gaugings, waterworks engineers had to arrive at the run-off in the best way they could, and the ctice before Parliamentary Committees had been to deduct from

Mr. Binnie. 14 to 16 inches from the average flow of the 3 driest consecuti years when questions of compensation water to the river were bei considered. A curve had recently been put forward which had show a direct relationship between rainfall and run-off, the differen between the two increasing with increasing rainfall. That mig

Fig. 21.



River Thames at Teddington weir:

Catchment-area 3,789 square miles.

Average annual rainfall (1883–1900) 26·38 inches.

,, ,, run-off ,, 8·15 ,, (30·9 per cent.)

Loss 18·23 ,,

River Severn at Bewdley:

Catchment-area 1,632 square miles.

Average annual rainfall (1921–1936) 38·14 inches.

,, ,, run-off ,, 18·90 ,, (49.6 per cent.)

Loss 19·24 ,,

be true if the facts were similar in regard to temperature, humidit and other questions affecting run-off at all places, but it w not true unless those conditions were exactly similar. In fac such records as there were of the relationship between rainfall a run-off in regions of low rainfall which were situated to the sou and east of England showed a much larger loss than places, say, the Highlands of Scotland, where there was a very high rainfall b

e conditions as to temperature and humidity were quite different. Mr. Binnie.
was not until records of gaugings of rivers and streams were ailable that it would be possible all over the country to deduce e run-off with any degree of certainty when the rainfall was known. rtunately, rainfall figures were given in *British Rainfall*, which had en started 77 years ago by the enterprise of a private individual, the e Mr. G. J. Symons; in 1919 the British Rainfall organization had en taken over by the Air Ministry, at first under the late Mr. M. de rle Salter and then under Dr. John Glasspoole. As a result of that terprise, excellent records of rainfall in Great Britain were available.

Sir Clement Hindley.
Sir Clement Hindley remarked that, as a member of the Inland ater Survey Committee, he might be able to give some information the history of water-survey in recent years. Further, as a member the Advisory Council of the Department of Scientific and Industrial search, he had seen something of the work of the Water Pollution search Board, which had for a long time been interested in the estion of water-survey. Before doing so, however, he would int out that a special debt of gratitude was due to Professor Dixon the work under discussion, extending over a period of 15 years, d he wished that Professor Dixon could have been present that ening.

The importance of water-survey had been urged on the Government a great many years. Commission after Commission had pointed t the lack of information in regard to water-resources throughout e country. One or two important steps might be mentioned in at connexion, and also one of the reasons why the Water Pollution search Board was so interested in the matter. The Royal Com-ssion on Sewage Disposal, which had sat from 1908 to 1915, had lled attention to the need for more accurate information in regard river-flow. They had pointed out that the degree to which it was cessary to purify sewage would depend on the dilution afforded by e river or stream into which the effluent was discharged, and that erefore it was of great importance that engineers and others con-cted with sewage-disposal should have information of that kind ailable. The particular side of water-survey connected with river-uging had been taken up actively by the Department of Scientific d Industrial Research. The Authors, on p. 82, called attention to e fact that the Departmental Committee on the Gauging of Rivers d Tidal Currents had really supervised the work at Bewdley for or 3 years, and that later the Department had supported it ancially. Two very distinguished Past-Presidents of The Institu-n, the late Sir Maurice Fitzmaurice, C.M.G., and the late Professor . C. Unwin, had served on that Committee, so that The Institution d been connected with the matter for a long time. The Govern-

Sir Clement Hindley.

ment had turned a deaf ear to most of the recommendations mad They had had the question of water-survey put before them as lon ago as 1902 by the Royal Commission on Salmon Fisheries, and agai by the Royal Commission on Sewage Disposal in 1908, as well as b the Canal and Waterways Committee of 1910, the Selected Committe on Water Supplies Protection Bill, 1910, and the Water Powe Resources Committee of 1921. More recently the British Associatio at their meeting at York had appointed a special research committe to inquire into the position generally, and in 1933–34 The Institutio and the British Association jointly had made a strong representatio to the Prime Minister, which had resulted in a deputation headed b Sir Henry Maybury, then President Inst. C.E., and by Sir Jame Jeans, then President of the British Association, being receive by the Minister of Health. The Inland Water Survey Committe had been appointed 2 years after that, and had, therefore, just starte on its third year of work; he was glad to see that Mr. Willian Allard was present that evening, so that he would be able to answe some of the questions which Mr. Binnie had raised. The Inlan Water Survey Committee was assisted by a very strong branch of th Ministry of Health, who were working whole-time on the subjec The work had really been divided up, broadly speaking, into thre aspects: rainfall, survey of surface-water, and survey of undergroun water. For the section on rainfall the Committee relied on th British Rainfall Association which, as Mr. Binnie had said, had fo many years made very complete records of rainfall. For th underground-water survey the Committee had the advantage c the services of the Geological Survey, and that work was bein pushed forward in a very thorough manner. The surface-wate survey depended on work that could be done by the Catchmen Boards, and it was in that direction that more work was require He thought that when the Committee's next Report was publishe and when the first Annual Year Book was produced, it would b seen how the problem was being tackled. It was bound to be ver incomplete at the start, because detailed and accurate records wer wanting in many places, but the Committee hoped that those gap would be gradually filled in.

He thought that the Paper was particularly interesting as giving specific instance of practical work of river-gauging—one of the ver few in Great Britain which might be said to be based on accurat and careful lay-out and scientific supervision. Another well-know example was the work of Mr. W. N. McClean. Those were out standing instances of what could be done if the Catchment Board could be persuaded to carry out the work systematically. Many c the Catchment Boards had records, and they were being mad

ailable to some extent, but a lot more work had to be done to stitute the systematic taking of records. He hoped that Mr. Allard ould be able to supplement what he had said and to give more tails of the work which was actually being carried out. Sir Clement Hindley.

Mr. William Allard, referring to Mr. Binnie's remarks, said at the intention was that the Statistical Year Book would be blished by the Inland Water Survey Committee, and would not published by the Ministry of Health as something independent om the Committee's reports. Mr. Allard.

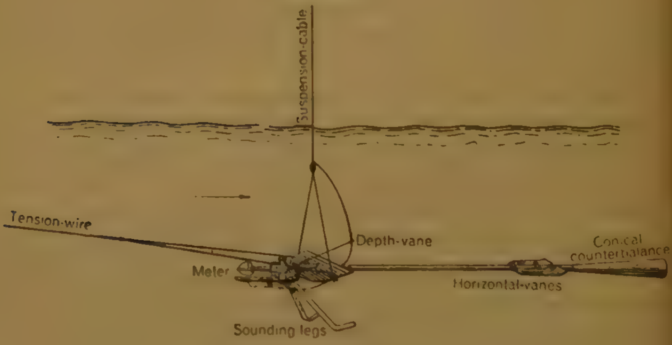
The Paper would be most useful to all who were likely to be conrned in the near future with establishing gauging-stations in Great itain, although, as it pointed out, the methods to be adopted for y station were bound to depend upon the features of the river at at point. The reasons given for selecting the Severn for study rved to illustrate the frequency with which English rivers were structed by weirs; they thereby offered to students of floweasurement in Great Britain several interesting problems to vestigate.

He had found, when taking current-meter measurements in the ver Tigris, that it was necessary to use a 120-lb. weight suspended om the meter. American river-surveyors considered that their velopment of stream-gauging equipment could be divided into ree periods: from 1888, with the inauguration of a definite uging-programme, to 1903, dealing with the improvement of an isting current-meter; from 1903 to 1920, dealing with the invention several water-level recording devices; and from 1920 to the present te, dealing with the development of equipment suitable for obtaing accurate soundings and velocity-measurements in deep swift ter. The Paper related to the third period. American practice peared to be to use as a sinker a heavy weight of as much as 150 lbs., t a well-known Bavarian instrument-maker had designed his eters so that the sinker could be added to the meter itself in such way as to maintain the latter's streamlined form, the added weight ing as large as 220 lbs. The use of such heavy weights had eated a corresponding need for a strong suspensory cable and for e combination of that cable with that which conveyed the electrical rrent, so as to reduce the disturbance of the stream and the drag the latter upon the cables. A suitable type of cable was made in eat Britain and had been used successfully for some years past in ypt and Iraq. He thought that it was of the same type as that that e Authors had shown in their lantern-slides of the Severn. With signs of cableway such as that used by Mr. McClean in recent years Great Britain, or those that the Bavarian maker constructed, the ole of the operation of the cableway could be effected from one

Mr. Allard. bank of the river without any trouble due to tangling such as th Authors mentioned. Such cableways were much heavier in al respects than the one used at Bewdley, whose portability might perhaps, sometimes be very convenient.

A noteworthy device for utilizing the pressure of flowing wate for causing the current-meter to sink was that of Herr Albrecht o the Bavarian Water Survey[1] (*Fig. 22*). The lengthy tail of th meter carried an inclined plate set at right-angles to the tail. Th current pressed upon the upper face of the plate and accordingl tended to depress the meter. Underneath the meter was a device fo indicating when bottom was touched by an electrical signal throug the cable. Unless retaining lines, which were often far from easy t arrange, were used in conjunction with the cable-suspended meter

Fig. 22.



ALBRECHT METER.

some drag of the suspension cable was bound to occur if the rive were running strongly. The Authors had indicated the metho used by them for correcting observed depths. The Egyptian method a reference to which they quoted, related the correction to be use for any observed depth to the mid-depth velocity (Table V). A American method made the correction depend instead upon the angl of declination of the cable from the vertical at its point of suspensio (Table VI, pp. 122 and 123).

The general constancy of the stage-discharge curve at Bewdley ha been demonstrated by the Authors, but similar conditions could no safely be assumed for all other British rivers, or even for other station on the Severn. Instances of variability of the relationship were know in the case of French and German rivers (*Figs. 23* and *24*, p. 124)

[1] "Der Tiefensteuer-Schwimmflügel." *Die Wasserkraft*, 1925 (Part II).

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TABLE V.—CORRECTIONS TO OBSERVED SOUNDINGS.[1]

(Conical weight 48 kilograms, suspension 1·6 millimetre diameter.)

Iid-depth velocity: ietres per second.	Observed sounding: metres.														
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
	Correction to be subtracted from observed sounding: centimetres.														
0·8	0	—	—	—	—	—	—	—	—	—	—	—	—	—	0
0·9	0	—	—	—	—	—	—	—	—	—	—	—	—	—	1
1·0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	1
1·1	0	0	0	0	0	0	0	0	0	0	1	1	1	1	2
1·2	0	0	0	0	0	0	0	0	0	0	1	1	2	2	2
1·3	0	0	0	0	0	0	0	0	0	1	1	2	2	3	3
1·4	0	0	0	0	0	0	0	0	0	1	2	2	3	4	4
1·5	0	0	0	0	0	0	0	1	1	2	2	3	4	5	6
1·6	0	0	0	0	0	0	1	1	2	2	3	4	5	7	8
1·7	0	0	0	0	0	1	1	2	2	3	4	5	7	8	9
1·8	9	0	0	0	0	1	2	2	3	4	5	7	8	10	12
1·9	0	0	0	1	1	2	2	3	4	5	6	8	10	12	14
2·0	0	0	0	1	1	2	3	4	5	6	8	10	12	15	17
2·1	0	0	0	1	2	2	4	5	6	7	10	12	14	18	20
2·2	0	0	0	1	2	3	4	6	7	9	12	14	17	21	24
2·3	0	0	1	2	2	4	5	7	8	10	13	17	20	24	28
2·4	0	0	1	2	3	4	6	8	10	12	16	20	24	28	33
2·5	0	0	1	2	3	5	6	9	11	14	18	22	27	32	38
2·6	0	0	1	2	4	6	7	10	13	16	21	26	31	37	43
2·7	0	0	1	3	4	6	8	12	15	19	24	29	35	42	49
2·8	0	1	2	3	5	7	10	13	17	21	27	33	39	47	56
2·9	0	1	2	4	6	8	11	15	20	24	31	37	44	53	63
3·0	0	1	2	4	6	9	13	17	22	27	34	41	49	59	70

[1] *Reproduced from* "Handbook of Instructions for Discharge Observers in gypt and the Sudan," p. 5. Cairo, 1929.

he Authors had shown (p. 94) how little was the comparative ifference in results that occurred if the discharges of three-month eriods were calculated in three very different ways. The same agree-ient of results would naturally not be obtained for shorter periods, as able VII (p. 125) showed. He had prepared that Table in order o see how far, even if harmony could be attained in long-period esults, there would be variation in the daily results. He had taken he records of three stations with drainage-areas of (X) 747 square iiles, (Y) 187 square miles, and (Z) 17 square miles. The discharges ad been worked out in the ways stated at the head of the Table.

Mr. Allard.

TABLE VI.—CORRECTIONS IN FEET TO REDUCE WET-LINE DEPTHS TO

Wet-line depth: feet.	Vertical angle in sounding line at							
	4	6	8	10	12	14	16	18
10	0·01	0·02	0·03	0·05	0·07	0·10	0·13	0·16
12	0·01	0·02	0·04	0·06	0·09	0·12	0·15	0·20
14	0·01	0·02	0·04	0·07	0·10	0·14	0·18	0·23
16	0·01	0·03	0·05	0·08	0·12	0·16	0·20	0·26
18	0·01	0·03	0·06	0·09	0·13	0·18	0·23	0·30
20	0·01	0·03	0·06	0·10	0·14	0·20	0·26	0·33
22	0·01	0·04	0·07	0·11	0·16	0·22	0·28	0·36
24	0·01	0·04	0·08	0·12	0·17	0·24	0·31	0·39
26	0·02	0·04	0·08	0·13	0·19	0·25	0·[illegible]3	0·43
28	0·02	0·04	0·09	0·14	0·20	0·27	0·36	0·46
30	0·02	0·05	0·10	0·15	0·22	0·29	0·38	0·49
32	0·02	0·05	0·10	0·16	0·23	0·31	0·41	0·52
34	0·02	0·05	0·11	0·17	0·24	0·33	0·44	0·56
36	0·02	0·06	0·12	0·18	0·26	0·35	0·46	0·59
38	0·02	0·06	0·12	0·19	0·27	0·37	0·49	0·62
40	0·02	0·06	0·13	0·20	0·29	0·39	0·51	0·66
42	0·03	0·07	0·13	0·21	0·30	0·41	0·54	0·69
44	0·03	0·07	0·14	0·22	0·32	0·43	0·56	0·72
46	0·03	0·07	0·15	0·23	0·33	0·45	0·59	0·75
48	0·03	0·08	0·15	0·24	0·35	0·47	0·61	0·79
50	0·03	0·08	0·16	0·25	0·36	0·49	0·64	0·82
52	0·03	0·08	0·17	0·26	0·37	0·51	0·67	0·85
54	0·03	0·09	0·17	0·27	0·39	0·53	0·69	0·89
56	0·03	0·09	0·18	0·28	0·40	0·55	0·72	0·92
58	0·03	0·09	0·19	0·29	0·42	0·57	0·74	0·95
60	0·04	0·10	0·19	0·30	0·43	0·59	0·77	0·98
62	0·04	0·10	0·20	0·31	0·45	0·61	0·79	1·02
64	0·04	0·10	0·20	0·32	0·46	0·63	0·82	1·05
66	0·04	0·11	0·21	0·33	0·48	0·65	0·84	1·08
68	0·04	0·11	0·22	0·34	0·49	0·67	0·87	1·12
70	0·04	0·11	0·22	0·35	0·50	0·69	0·90	1·15
72	0·04	0·12	0·23	0·36	0·52	0·71	0·92	1·18
74	0·04	0·12	0·24	0·37	0·53	0·73	0·95	1·21
76	0·05	0·12	0·24	0·38	0·55	0·74	0·97	1·25
78	0·05	0·12	0·25	0·39	0·56	0·76	1·00	1·28
80	0·05	0·13	0·25	0·40	0·58	0·78	1·02	1·31
82	0·05	0·13	0·26	0·41	0·59	0·80	1·05	1·34
84	0·05	0·13	0·27	0·42	0·60	0·82	1·08	1·38
86	0·05	0·14	0·28	0·43	0·62	0 84	1·10	1·41
88	0·05	0·14	0·28	0·44	0·63	0·86	1·13	1·44
90	0·05	0·14	0·29	0·45	0·65	0·88	1·15	1·48
92	0·06	0·15	0·29	0·46	0·66	0·90	1·18	1·51
94	0·06	0·15	0·30	0·47	0·68	0·92	1·20	1·54
96	0·06	0·15	0·31	0·48	0·69	0·94	1·23	1·57
98	0·06	0·16	0·31	0·49	0·71	0·96	1·25	1·61
100	0·06	0·16	0·32	0·50	0·72	0·98	1·28	1·64

The observed depth, or the depth indicated by a reel at some distance above
shown in the above Table are applied.

The corrections should be used to the nearest tenth of a foot; hundredths
Corrections for a wet-line depth of 100 feet are numerically equal to a per-

[1] From report of Mr. F. C. Shenehon in Annual Report of

Mr. Allard.

ctual Depths for Vertical Angles between 4 and 36 Degrees[1].

above water-surface: degrees.									Wet-line depth: feet.
20	22	24	26	28	30	32	34	36	
0·20	0·25	0·30	0·35	0·41	0·47	0·54	0·62	0·70	10
0·24	0·30	0·36	0·42	0·49	0·57	0·65	0·74	0·84	12
0·29	0·35	0·41	0·49	0·57	0·66	0·76	0·87	0·98	14
0·33	0·40	0·47	0·56	0·65	0·76	0·87	0·99	1·12	16
0·37	0·45	0·53	0·63	0·73	0·85	1·98	1·12	1·26	18
0·41	0·50	0·59	0·70	0·82	0·94	1·09	1·24	1·40	20
0·45	0·55	0·65	0·77	0·90	1·04	1·20	1·36	1·54	22
0·49	0·60	0·71	0·84	0·98	1·13	1·31	1·49	1·68	24
0·53	0·64	0·77	0·91	1·06	1·23	1·41	1·61	1·81	26
0·57	0·69	0·83	0·98	1·14	1·32	1·52	1·74	1·95	28
0·61	0·74	0·89	1·05	1·22	1·42	1·63	1·86	2·09	30
0·65	0.79	0·95	1·12	1·31	1·51	1·74	1·98	2·23	32
0·69	0·84	1·01	1·19	1·39	1·60	1·85	2·11	2·37	34
0·73	0·89	1·07	1·26	1·47	1·70	1·96	2·23	2·51	36
0·78	0·94	1·12	1·33	1·55	1·79	2·07	2·36	2·65	38
0·82	0·99	1·18	1·40	1·63	1·89	2·18	2·48	2·79	40
0·86	1·04	1·24	1·47	1·71	1·98	2·28	2·60	2·93	42
0·90	1·09	1·30	1·54	1·80	2·08	2·39	2·73	3·07	44
0·94	1·14	1·36	1·61	1·88	2·17	2·50	2·85	3·21	46
0·98	1·19	1·42	1·68	1·96	2·27	2·61	2·98	3·35	48
1·02	1·24	1·48	1·75	2·04	2·36	2·72	3·10	3·49	50
1·06	1·29	1·54	1·82	2·12	2·45	2·83	3·22	3·63	52
1·10	1·34	1·60	1·89	2·20	2·55	2·94	3·35	3·77	54
1·14	1·39	1·66	1·96	2·28	2·64	3·05	3·47	3·91	56
1·18	1·44	1·72	2·03	2·37	2·74	3·16	3·60	4·05	58
1·22	1·49	1·78	2·10	2·45	2·83	3·26	3·72	4·19	60
1·26	1·54	1·84	2·17	2·53	2·93	3·37	3·84	4·33	62
1·31	1·59	1·89	2·24	2·61	3·02	3·48	3·97	4·47	64
1·35	1·64	1·95	2·31	2·69	3·12	3·59	4·09	4·61	66
1·39	1·69	2·01	2·38	2·77	3·21	3·70	4·22	4·75	68
1·43	1·74	2·07	2·45	2·86	3·30	3·81	4·34	4·89	70
1·47	1·79	2·13	2·52	2·94	3·40	3·92	4·46	5·03	72
1·51	1·84	2·19	2·59	3·02	3·49	4·03	4·59	5·17	74
1·55	1·88	2·25	2·66	3·10	3·59	4·13	4·71	5·30	76
1·59	1·93	2·31	2·73	3·18	3·68	4·24	4·84	5·44	78
1·63	1·98	2·37	2·80	3·26	3·78	4·35	4·96	5·58	80
1·67	2·03	2·43	2.87	3·35	3·87	4·46	5·08	5·72	82
1·71	2·08	2·49	2·94	3·43	3·96	4·57	5·21	5·86	84
1·75	2·13	2·55	3·01	3·51	4·06	4·68	5·33	6·00	86
1·80	2·18	2·60	3·08	3·59	4·15	4·79	5·46	6·14	88
1·84	2·23	2·66	3·15	3·67	4·25	4·90	5·58	6·28	90
1·88	2·28	2·72	3·22	3·75	4·34	5·00	5·70	6·42	92
1·92	2·33	2·78	3·29	3·84	4·44	5·11	5·83	6·56	94
1·96	2 38	2·84	3·36	3·92	4·53	5·22	5·95	6·70	96
2·00	2·43	2·90	3·43	4·00	4·63	5·33	6·08	6·84	98
2·04	2·48	2·96	3·50	4·08	4·72	5·44	6·20	6·98	100

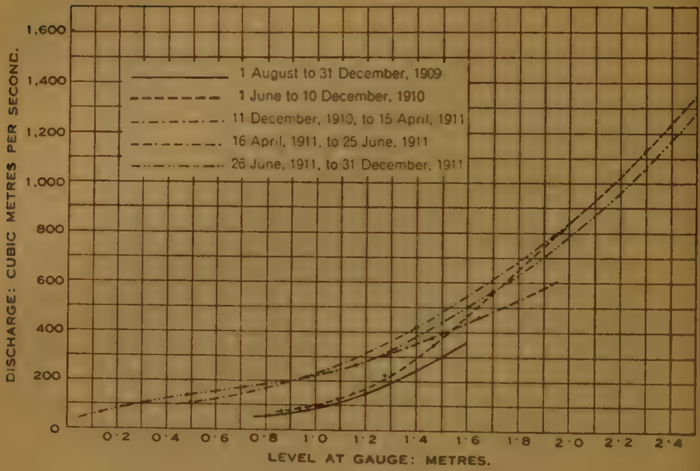
he water-surface, must be reduced to the wet-line depth before the corrections

re given to aid in interpolation for odd degrees and depths.
entage correction for any depth.

he Chief of Engineers, U.S. Army, 1900 (Part VIII), p. 5330.

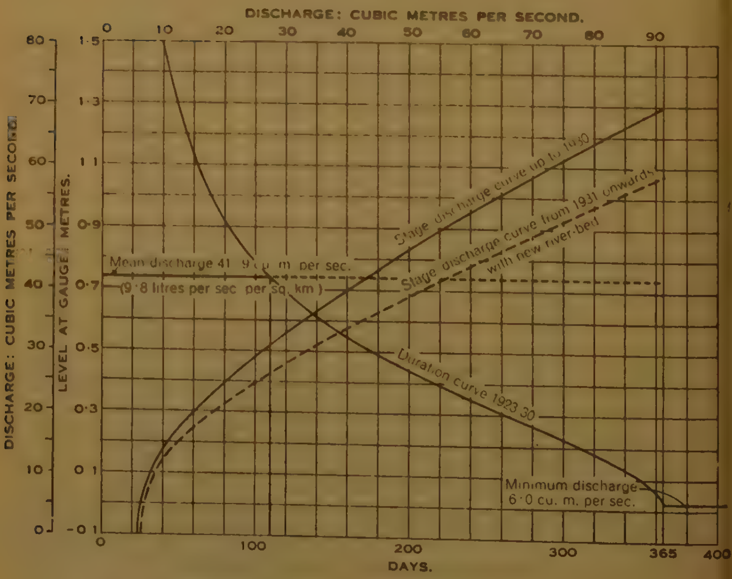
Mr. Allard

Fig. 23.



Stage–Discharge Curves for River Durance at Mirabeau.

Fig. 24.



Stage–Discharge Curves for River Neckar at Münster.

e reason that the hours chosen in the case of B were as shown, and Mr. Allard.
t at regular intervals throughout the 24 hours, was that they were
nes at which people could be expected to read the gauge without
ry much trouble. The differences between daily discharges as
tained by methods A and C, and, alternatively, by methods B
d C, had been worked out in terms of percentage, plus or minus,
d the monthly means of those percentages had been calculated.

BLE VII.—COMPARISON OF RESULTS OBTAINED FROM CONTINUOUS DISCHARGE-RECORDS.

denotes the result calculated from the discharge at 9 a.m.
,, ,, ,, ,, ,, mean of discharges at 8 a.m. and 4 p.m., or, with high stages, at 8 a.m., noon, 4 p.m. and 8 p.m.
,, ,, ,, ,, ,, mean of continuous record for 24 hours, midnight to midnight.

	Percentage difference between A and C.		Percentage difference between B and C.	
	Maximum for single day.	Monthly mean of percentages.	Maximum for single day.	Monthly mean of percentages.
ation X				
May, 1936	− 55	− 1	− 22	+ 10
June	− 71	− 2	− 47	− 2
July	− 59	+ 1	− 35	− 1
ation Y				
Nov., 1935	− 63	− 8	− 33	+ 2
Dec.	− 51	− 1	− 30	+ 1
Jan., 1936	− 61	− 8	− 32	− 2
ation Z				
May, 1935	− 64	− 24	+ 29	+ 2
June	+ 171	− 6	+ 39	+ 7
July	− 48	− 10	− 69	+ 3
Nov., 1935	− 84	+ 2	+ 24	+ 5
Dec.	+ 92	− 8	− 41	+ 3
Jan., 1936	+ 89	− 3	+ 43	− 3

uch monthly means would be seen to be at times quite small, like
e differences in results obtained by the Authors. Nevertheless,
owever closely the monthly means of results obtained by two
ifferent methods might agree, the daily differences between them,
shown by Table VII, ranged from 22 per cent. to 171 per cent. in
ose particular cases. Those remarks were not intended to contra-
ict anything the Authors had said, but to remind those who set up
auging-stations that they had to consider (having in mind the
sults they were trying to obtain) whether they could be satisfied

Mr. Allard. with one reading a day, or whether they would be well advised to make a continuous record, if they could manage to provide the instrument.

The second of the Authors' three methods was to calculate the daily discharge from the mean of four or eight water-levels observed during that day. Although that method was considered suitable for a stream which rose and fell very slowly [1], Table VIII showed the

TABLE VIII.—COMPARISON OF RESULTS OBTAINED BY ESTIMATING OCCASIONAL 24-HOUR DISCHARGES OF A STREAM OVER A CLEAR-OVERFALL WEIR FROM:—

A. Mean level for 24-hour period.
B. Continuous record of rates of discharge.

Item.	Daily discharge as estimated from		Quantity by which A falls short of B: per cent.	Remarks as to water-levels during period.
	A.	B.		
1	38·0	38·1	—	Small rise and fall.
2	12·0	12·1	1	Nearly horizontal.
3	7·0	7·1	2	Small rise and fall.
4	6·0	6·2	3	Small rise and fall.
5	6·2	6·4	3	Moderate fall and rise.
6	28·0	29·9	6	Moderate fall.
7	9·0	9·8	8	Small rise, moderate fall.
8	9·6	10·5	9	Moderate fall.
9	15·0	17·0	12	Steep fall.
10	17·5	20·1	13	Steep rise, fall and rise.
11	53·0	61·6	14	Steep rise.
12	23·5	28·2	17	Steep fall and rise.
13	60·0	72·9	18	Steep rise and fall.

errors introduced by using it in the case selected, which related to the run-off of an area of 5,000 acres. Thirteen tests had been made. The discharge had been worked out by two methods: first by taking twelve observations of level during the 24 hours, taking the mean of those levels, and then calculating the discharge from that level according to the rating curve for that station. The other method had been to take the mean of twelve measurements of discharge made at 2-hour intervals during the 24 hours. It would be seen that the quantity by which A fell short of B ranged from 0 to 18 per cent., the percentage depending upon the total range of level during the day. Again, that was not intended to contradict anything the Authors had said. It was, perhaps, a continuation of an earlier discussion on the subject.[1]

[1] *Discussion on* "Practical River Flow Measurement and its Place in Inland Water Survey, as exemplified on the Ness (Scotland) Basin," by W. N. McClean. Trans. Inst. W.E., vol. xxxviii (1933), p. 269.

It showed that, whilst with a quiet river, or one that was not rising Mr. Allard.
ıd falling substantially on the day that was being considered, it was irly safe to take the mean level as an index of the mean discharge, ıce the river got "lively" considerable errors might arise if the me were being done. The foregoing remarks applied to a catchent of 5,000 acres, whereas the Authors' catchment was 1,600 square iles, and he was not suggesting that the same figure of 20 per cent. ror would apply to any size of area or any other river.

Referring to the question of relating rainfall and run-off to the lendar year and the water year, apparently it made very little fference which method was used as far as the Authors were conrned; much the same result was obtained. It might be interesting engineers to know that the Inland Water Survey Committee had nsidered what 12 months formed the most suitable water year, ıd in doing so they had also taken into account underground water ıd the Meteorological Office's studies of seasonal rainfall; they mained of the opinion that 1st October to 30th September was the ost suitable 12 months. He did not think the Authors were ggesting an alternative to that in what they said.

Mr. FRANK HIBBERT said that it was interesting to note that, whilst Mr. Hibbert.
e work had been commenced for scientific rather than for economic asons, the latter aspect was already recognized by its value to the vern Catchment Board. The Inland Water Survey Committee so appreciated its worth.

On p. 84 reference was made to the effect of the Vyrnwy impounding orks of the Liverpool Corporation on the discharge at the Bewdley uging-station, and, whilst it was agreed that the total annual flow ıs affected to the extent of less than 1 per cent., the effect of the servoir in times of heavy rain must have been to smooth out to me extent the peak flows. The catchment-area above the Vyrnwy ım was approximately $2\frac{1}{4}$ per cent. of the total area above Bewdley, ıd the long-period average rainfall at Vyrnwy was approximately 0 per cent. of the long-period average of the Bewdley catchment. dry periods there would be an increased and more regular flow as result of the discharge of compensation water at Vyrnwy, and it ould appear that, in an exceptionally dry period (for example, ly, 1934), the total compensation water paid out had averaged out 10 per cent. of the flow at Bewdley during the driest 14 days. *Figs. 12* and *13* (pp. 98 and 99) were two flood-hydrographs which owed entirely different characteristics, and he thought it would have ded interest to the Paper if the Authors had suggested some reason that difference. An examination of the conditions prevailing on gh ground at the time of the floods—taking the Vyrnwy records as presentative—might throw some light on their special features.

Mr. Hibbert.

Fig. 12 showed a rapid rise on the 1st June, a sustained peak to the 3rd June, and then a gradual fall. At Vyrnwy, the rainfall for the 4 days, 30th May—2nd June, 1924, had been 2·36 inches, a low rainfall compared to the 3 to 4 inches for the whole of the Severn Valley, 31st May—1st June. The bulk of the rain at Vyrnwy had fallen in the 24 hours from 8 a.m. on the 31st May to 8 a.m. on the 1st June, and the maximum run-off had been recorded in the same 24 hours.

Before the heavy rainfall, the reservoir had been at overflow-level, but had backed up and retained the peak run-off. The maximum discharge to the river had been on the 1st June. Of the 2·36 inches of rainfall, the run-off in the period, the 31st May to the 7th June, had been about 84 per cent., and, by the 7th June, about 68 per cent. had been discharged to the river. The run-off for the two days, 31st May and 1st June, had been about 50 per cent. of the rainfall.

A possible explanation of the rapid rise of the river at Bewdley was:—

(1) The rainfall had been heaviest on the lowland areas.
(2) The flood had occurred at a time when the river-discharge had been low following a period of low rainfall.

The sustained peak and gradual fall could be accounted for by the prolonged period of heavy rain on the high ground. At Vyrnwy, the rain had continued for more than a week after the flood, when over 2 inches of rain were recorded, whilst, according to the Authors, the days following the 1st June had been fine.

Comparing the run-off and rainfall for the year 1923–24, it would be seen from Table II (p. 104), that February and March, 1924, had been months of low rainfall, whilst the run-off for the 3 months, February, March, and April (Table I, p. 102), was only 43·2 per cent. of the rainfall, against a normal 67·2 per cent. The rainfall figures for May (Table II) included the 3 to 4 inches on the 31st May, and, deducting say 3 inches for that day, the remainder of the May rainfall had been about normal.

Continuing the comparison of rainfall and run-off, the period of May and June had had a rainfall of 8·58 inches (Table II) and a run-off of 4·64 inches (Table I), or a run-off of 54·1 per cent. against a normal 37 per cent., whilst June alone had had 90·4 per cent. run-off against a normal 34·2 per cent. The rainfall for June had been about 15 per cent. above normal. The high percentage of run-off in June would appear to indicate that a greater percentage of the rain of the 31st May to the 1st June had been ultimately discharged into the river. The suggestion that 52 per cent. had been probably lost did not appear to be substantiated by reference to the figures for loss (Table III, p. 106). The total loss for the year 1923–24 was about normal.

Fig. 13 showed a gradual rise and sudden fall. At Vyrnwy, the nfall for the 3 days from 14th to the 16th February, 1928, had been ; inches, the bulk of the rain having fallen on the 15th February, e maximum run-off having been recorded the same day. Of the 4·8 :hes of rain, the run-off in the period from 16th to the 22nd February d been about 89 per cent., and, in the same period 82 per cent. had en discharged to the river, 69 per cent. having been discharged in the lays from 15th to the 17th February. After the heavy rain of the th to the 16th February, it had continued fine until the end of the onth, frost setting in on the 20th February and continuing into the ddle of March. The temperature at the time of the heavy rain d been 45° F. Mr. Hibbert.

Widespread frost throughout the Severn Valley, and especially the high ground, would account for the sudden drop in the discharge Bewdley. The Tables of rainfall and run-off showed that for the riod from January and February, 1928, the rainfall at Bewdley had en 52 per cent. above normal, and the run-off 73 per cent. above rmal, the run-off for January and February being 93 per cent. of e rainfall, which was a very high proportion.

In March, the rainfall had been 17 per cent. above normal, and the n-off 16 per cent. below normal, the run-off being 56·5 per cent. of e rainfall. In April, the rainfall had been 31·6 per cent. below rmal, whereas the run-off had been 13 per cent. above normal, the n-off being 78·5 per cent. of the rainfall against the normal 47·4 r cent. for April.

The high percentage run-off during January and February (that s, 93 per cent.), and the small percentage run-off in March, namely 56 r cent., followed by a high percentage run-off in April of 78 per cent., uld suggest that some part of the heavy rainfall of February had en held up by the frost which had persisted from the 20th February the middle of March.

He would like to know whether any difficulty had been experienced maintaining the gauge-readings during periods of heavy frost, there was evidence to show that in connexion with gaugings ken on the Severn at Worcester in 1811, the river had been bound.

Mr. Harry Chapman said that his remarks would be in the nature a comparison between the methods of gauging employed by the thors of the Paper and those by Mr. McClean and himself in the ensive gauging carried out by Mr. McClean during the last rears, and also in 1912–13, of certain Scottish rivers, including a y complete survey of the Ness area, the Dee and the upper Spey. might add that some 300 gaugings had been made, the measured ws ranging from 50 cusecs to 13,000 cusecs. Mr. Chapman.

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Mr. Chapman. As none of the sites had been readily accessible from London, the policy had been to aim at gauging the whole range of levels at on visit of 2 or 3 months in the autumn and spring, as floods might b expected then and also frost on the hills would probably occur an low flows would result. In some cases where the work had not bee completed, the very little left to be done had been carried out b local engineers when conditions had been suitable. The gaugin apparatus had accordingly been designed for use by night as well a by day and observations could be, and had been, carried on con tinuously throughout the whole period of a flood from the beginnin of a rise until the river had fallen again.

The first gaugings, on the river Garry in 1913, had been carrie out by means of a ropeway and travelling seat from which th observer took readings by means of a current-meter on a rod. Th ropeway had been fixed to convenient trees and the traveller ha been worked by a second observer on the bank who had also booke the signals given by the current-meter. That method had bee fairly satisfactory up to a certain point, but it had been found im possible to manipulate the rod when the velocity of flow was high

It had been considered, however, that it was desirable, if possibl to fix the direction of the meter at right angles to the section, and s the next step had been to use a punt or boat, anchored to a travelle on a ropeway and steadied by a cross-rope, with the meter on a fram over the bows. A substantial wire ropeway had been slung fro trestles, one on either bank, and the traversing of the boat had bee by means of a winch attached to one of the trestles. The mos recent boat had been the double punt, fitted with a cabin for th observer from which the current meter could be set at any desire depth below the surface and the position of the boat could b adjusted transversely and longitudinally as required. The meter-ro had been streamlined in section and made strong enough to work i high velocities at considerable depths. The boat had proved ver stable, and very regular and uniform results had been obtaine but, as it had been a punt and not boat-shaped, there ha been limits to the velocity of flow in which it had been possibl to operate it.

Attention had therefore been directed to the use of a simila apparatus to that which had been used on the Severn, and Professo Dixon had very kindly allowed him to examine his gear and, in fact there had been incorporated in the apparatus some of Professo Dixon's ideas, particularly that of measuring the depth of the mete below the water-surface by means of a spring tape.

The apparatus was worked entirely from one bank by means of a double winch on the standard and an endless traversing rope. Th

ıch was capable of traversing the meter in a horizontal direction, Mr. Chapman. d, having reached the desired distance from the initial point, the ter might be raised or lowered as required.

The matter of the reduction of drift had been dealt with as far as ssible by using a streamlined sinker and the finest possible sup-rting wire, which was also the transmitting wire for the contact nals. It was a stranded steel wire, ¼ inch in circumference minally, but it had bedded into its core until it was even less ın ¼ inch. Experiments in the National Physical Laboratory tank owed that the supporting wire was very largely responsible for the ft, and by reducing that as far as possible the displacement of the ter had apparently been reduced to such an extent that the ference between the actual and theoretical soundings was gligible, except, possibly, during high floods.

The method of sounding, perhaps, deserved a word or two. For ne time a level had been used, a light staff being fixed to the wire pporting the meter above the sinker, and that was traversed across m point to point, lowered until the weight rested on the bottom, d the level read off as in surveying on land. That method had oved to be very successful and quick, and its advantages were that e staff was held perfectly vertical and in the exact spot where the ight would rest during gauging, both of which were difficult to ain when a boat was used for placing the staff or sounding line. was found that soundings were repeated exactly during gauging, period of low water being chosen for doing the work.

Although gauging by means of the suspended meter had been the thod employed for some time, results had been persistently checked far as possible by using a rod meter on fixed supports, such as ntilevers, so that obliquity of flow might be allowed for. Arising t of it, some success had been achieved with the rod meter sup-rted on the cableway, and there was a reasonable hope that with tain alterations to the existing apparatus that method of gauging ght supersede the free-meter method.

Most of the rivers which had been gauged in Scotland differed nsiderably from the Severn in the time in which noticeable changes water-level took place. Except at very low levels, it was possible to arrive at an average for the day which was satisfactory m the gauging point of view. That being so, it had been the rule to serve water-levels at frequent intervals—in some cases every arter of an hour—and to plot those on a time base. The time of the y at which each section was being worked was also noted on the uging-sheets so that the water-level at that time might be identi-d, and it was therefore possible to plot a stage-discharge curve for ch vertical, or the flow per foot of width. From the curve, the

Mr. Chapman. flow at any desired water-level might be read off, and the stag discharge curve for the whole river was arrived at by reading o from the several curves the individual discharges at intervals o say, 0·1 or 0·2 foot, as required.

Water-levels were carefully observed by means of wells and a hook stick. The well might be only a temporary wooden box, fed by a pipe leading out from the bank. Slope-gauges had also been used and they were very satisfactory, especially for permanent sites. . stilling box should be used for such gauges to eliminate surfac movement.

Water-slopes in the gauging-reach and beyond were included i the records and temporary gauges were suitable for the purpose That was looked upon as a very important part of the work and it wa unfortunate that water-slope records were not, as a rule, availabl when choosing a site for current-meter measurement, as such record gave a very clear indication as to how the river was likely to behav at varying levels.

All the meters had been adapted for use either on a rod or sus pended on a wire. The 1913 gaugings had been made with tw Amsler meters, which had two-bladed propellers and were still in use In 1930 two more Amsler meters of a later pattern had been purchase and a Watt bucket-wheel-type meter had been bought about the sam time. An English meter made by Messrs. Cooke, Troughton an Simms had been loaned for testing and was subsequently purchase That was a very good meter having a four-bladed propeller, pro ducing an instrument in which the driving force was high compare with mechanical friction. That was a good point, and it was un fortunate that the firm had discontinued its manufacture, as wit certain small modifications it would have been a really excellen meter.

All the meters were tested at frequent intervals—after ever survey, if they were much used—in the National Physical Labora tory tank and, without exception, they had adhered very closely t their original ratings. It was correct to say that the variations ha been of no practical importance, as far as could be judged fror the tests. It was rather unfortunate that the tests were no entirely satisfactory for slow speeds; only one or two points coul be obtained on the curve below about 1 foot per second. Tha was the most difficult part of the curve to define as it wa at about 1 foot a second that the curve began to depart from straight line.

A small number of surface measurements had been made wit floats, but generally speaking they were not very suitable for use i Scottish rivers, on which it was difficult to find suitable reaches fo

e work. Furthermore, the results, even if they were reliable, ve average velocities over a distance rather than the velocity at a int. Mr. Chapman.

When working out the gauging-results, the ratio of the surface locity, as measured, to the mean velocity in each vertical, s always noted, and it had been found that the coefficients so tained had been very irregular in almost every case. It was, erefore, very difficult to work out the discharge from surface easurements and little confidence could be placed in the results. As rough indication, however, surface-floats no doubt had their uses d advantages.

Dr. JOHN GLASSPOOLE said that a study of the distribution of nfall, both in time and space, seemed of fundamental importance an understanding of records of stream-flow. In the past it was ar that the impetus for greater precision in the evaluation of rain-l had come more especially from engineers. It was when rain-l was considered in terms of run-off that precision was especially cessary. With an increase in the number of records of stream-w it seemed likely, therefore, that an even greater precision in the rious determinations of the volume of rainfall would be required, d in order to emphasize that point, reference was made to various mputations by Mr. E. G. Bilham, which he had been asked to ention. Dr. Glasspoole.

Mr. Bilham had found that the correlation-coefficient of the nual values of rainfall and run-off given in Tables II (p. 104) d I (p. 102) of the Paper was $r = 0{\cdot}92$. That signified that about hs of the variations of annual run-off were accounted for by riations of the annual rainfall as a single factor, leaving only $\frac{1}{6}$th the variations from the means to cover casual errors of observa-n, the differences in the storage in the area at the end of each year, e seasonal incidence of the rainfall and the other meteorological tors mentioned by Mr. Binnie. In view of the dominating relation-ip between annual values of rainfall and run-off, it was clear that, order to study the other factors, precise estimates both of rainfall d run-off were required.

Mr. Bilham had also given an equation connecting the annual lues of rainfall and run-off in inches, which was :—

$$\text{run-off} = \tfrac{2}{3}\,(\text{rainfall}) - 6{\cdot}6$$

ing that equation and the annual rainfall amounts only, the mputed values for the run-off were usually within 2 inches of the served values, as shown in Table VIII (p. 134).

Dr. Glasspoole. For the Thames valley above Teddington, the corresponding equation was :—

$$\text{run-off} = \tfrac{2}{3}\,(\text{rainfall}) - 9{\cdot}1$$

In that connexion it was important to note that similar equations had been shown to fit the observed facts reasonably well for those two different catchment-areas.

The relationships given above were between annual values of rainfall and run-off. Mr. Bilham had also considered the relationship between the variations in the rainfall and run-off during the winter and summer half-years separately. The corresponding correlation coefficients were 0·95 and 0·89 respectively. While that showed a somewhat closer relationship in the summer than in the winter, the differences in the correlation-coefficients were statistically hardly significant.

TABLE VIII.

Year.	Observed run-off: inches.	Run-off calculated from annual rainfall: inches.	Error: inches.
1921–22	16·1	18·1	+ 2·0
1922–23	18·6	17·4	— 1·2
1923–24	22·1	22·7	+ 0·6
1924–25	20·4	20·1	— 0·3
1925–26	15·7	17·7	+ 2·0
1926–27	20·7	23·6	+ 2·9
1927–28	23·2	18·9	— 4·3
1928–29	13·7	13·3	— 0·4
1929–30	26·6	27·4	+ 0·8
1930–31	26·3	24·7	— 1·6
1931–32	19·7	19·0	— 0·7
1932–33	15·3	12·8	— 2·5
1933–34	8·5	10·9	+ 2·4
1934–35	14·3	16·3	+ 2·0
1935–36	22·6	21·6	— 1·0
Average (regardless of sign)			1·65

The correlation-coefficient between the rainfall in the winter half-year and the run-off in the following summer half-year was only 0·18. That signified that the variations in the run-off from the mean during the summer were mainly dependent on the rainfall of that summer and only to a very slight extent upon the rainfall in the preceding winter half-year.

In spite of the close relationship between rainfall and run-off, it

ıs obvious that neither measurement could be dispensed with, Dr. Glasspoole. d in fact they could usefully supplement each other. Run-off ta were of particular service during periods of snow, when the rain-uge might not always function properly, whilst the rainfall records ere of especial use during floods when precise measurement of the w presented difficulties. In view of the close relationship between nual run-off and rainfall data over that area it was clear that the nual rainfall amounts, which could be obtained back to, say, 80, might be used to give estimates of the run-off over a much nger period. Before that was done it seemed desirable, however, examine the seasonal incidence of the rainfall and other factors, pecially in those years where there were differences between the lculated and observed values for the run-off.

Dr. Glasspoole hoped that the most useful piece of work which the ıthors had undertaken would be repeated in other areas, and that would stimulate further study, so that generalizations might be rmulated as to the rainfall and the resultant flow in the streams. seemed that real progress and a substantial contribution to ıowledge would be made along such lines.

Mr. C. H. ROBERTS remarked that the Paper formed a model Mr. Roberts. ' how the flows of rivers might be investigated. During the years ı11 to 1915 he had investigated the flow of several rivers in Scotland in particular that of the river Dee—for the purpose of a Bill in ırliament. He had afterwards read a Paper on the river Dee before e Institution of Water Engineers.[1] A comparison of some of e results that he had obtained with those described in the present ıper might be of interest, especially seeing that the average rainfall ı the Dee catchment was very nearly the same as that on the vern.

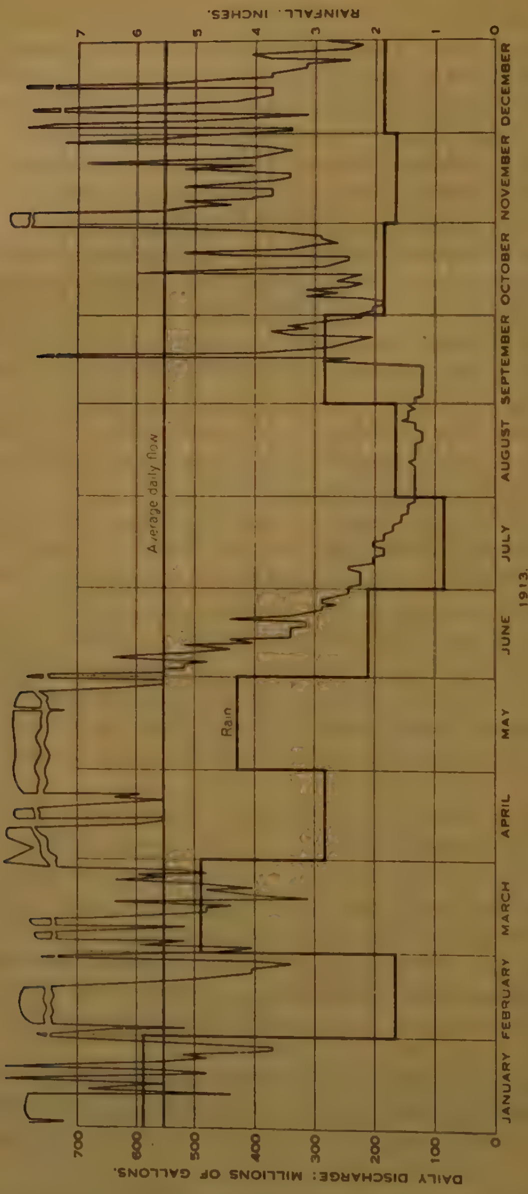
The river Dee rose in the Cairngorm mountains, was a fast-running ver, and had a catchment-area of 530 square miles above the ıuging station. About one-third of the area consisted of granite asses, and there were large glacial deposits and much decomposed ısorbent rock in the valleys and overlying the solid granite. There ere several lochs of considerable size in the area, but no weirs. hydrograph similar to those given in *Figs. 7* to *11* was shown in *ig. 25* (p. 136), and gave some indication of the character of the flow ' the Dee.

The rainfall and average flow during each month were plotted, ıd the hydrograph showed that the flow of the river responded more ıickly to rainfall than it did in the case of the Severn, as well as

[1] "Investigation of the Flows of the River Dee, Scotland." Trans. Inst. W.E. ıl. xxiv. (1919), p. 60.

Mr. Roberts.

Fig. 25.



Flow of River Dee at Cairnton Gauging-Station.
(Catchment-Area 530 Square Miles).

ffering in many other respects from that of the Severn. Some of Mr. Roberts. ose differences were, no doubt, due partly to the lesser drainage-ea of the Dee above the point of measurement (that was to say, 0 square miles as compared with 1,632 square miles), partly to the eeper profile of the river-bed, and partly to the different physical ture of the catchment-area. The discharge-curve of the Dee as concave instead of convex as it was in the case of the Severn *'ig. 5*, p. 91).

With reference to the recording of the results of observations, he reed with the Authors' view (p. 101), that for a Paper such as that ider discussion it was desirable for quantities to be stated in inches the catchment-area so that a direct comparison might be made

TABLE IX.—RIVER DEE BASIN (CATCHMENT-AREA 530 SQUARE MILES): YEARLY PRECIPITATION, DISCHARGE, EVAPORATION, ETC.

Col. (1).	Col. (2).	Col. (3).	Col. (4).	Col. (5).	Col. (6).
Year.	Precipitation: inches.	Total observed discharge: inches.	Evaporation, absorption, etc. (difference of Cols. (3) and (2): inches.	Assumed flood-discharge (from diagrams): inches.	Assumed percolated or underground water (from diagrams): inches.
11	34·3	23·3	11·0	11·0	12·3
12	41·1	29·2	11·9	13·0	16·2
13	33·3	25·7	7·6	11·2	14·5
14	38·9	25·1	13·8	13·0	12·1
15	44·1	28·7	15·4	13·9	14·8
erages	38·3	26·4	11·9	12·4	14·0

th results obtained from other rivers. If, in addition, the results ere plotted on a diagram showing the rainfall and the flow, a good al of very useful information would be obtainable, such as the fferent effects of rainfall on the flow at different times of the year d in all kinds of weather, the time-lag of increase of flow in con-xion with the rainfall, the loss at all times, etc. He joined with the ithors in suggesting that the starting of the water year should not the 1st January, but should be some time in the late summer or rly autumn.

Table IX gave a summary of the results obtained by him from the e basin for comparison with the results described in the Paper.

The average rainfall of 38·3 inches in the Dee catchment-area the period of the measurements had been lower than the mean

Mr. Roberts. rainfall over 35 years (which had been 40 inches), as estimated by Dr. Hugh R. Mill, but it happened to be very nearly the same as that in the Severn catchment-area, namely 38·14 inches, and that facilitated comparison. Other results, however, differed considerably. For instance, in the Dee area the average annual discharge was 26·… inches, or 68·9 per cent. of the precipitation. In the Severn area it was 19·9 inches, or a little less than 50 per cent. of the precipitation, which meant that nearly 20 inches were lost in evaporation, absorption, etc.; that was a high figure in comparison with the Dee, in which it was less than 12 inches.

With regard to flood-discharges, a hydrograph of the greatest flood in the Dee, which occurred in 1913 (*Fig. 25*, p. 136), approximated in form to that given for the Severn (*Fig. 12*, p. 98). The flood-discharge of the Dee in 1913 had arisen to a rate of 33 cusecs per 1,000 acres. The rise had been remarkably rapid, due to the path of the rainstorm following the direction of the flow of the river. That discharge compared with the highest flood-discharge of the Severn given in the Paper of 18,200 cusecs, or 17·5 cusecs per 1,000 acres. The lower figure in the case of the Severn was doubtless due in part to the greater catchment-area of that river.

With regard to minimum flows, the Severn was given as 200 cusecs, which was equivalent to 0·20 cusec per 1,000 acres. The lowest flow of the Dee had never fallen below 0·65 cusec per 1,000 acres. In order that the gauging of any river might accurately represent the flow, it was important that the position of the gauging-station should be selected where there was likely to be little or no flow in porous material under the bed of the river or in the banks, as any such flow would not be included in the gaugings. If it were not practicable to select such a position then some attempt should be made to estimate the underground flow in the manner recommended by Slichter of the American Geological Survey.[1] He had had the responsibility many years ago of investigating the flow of the river Salado in South America. That river had its origin on the eastern slopes of the Andes, and the character of the flow differed very much from that of both the Dee and the Severn. In particular, it might be mentioned that the flow of one of its tributaries ceased altogether for several weeks in the hot season owing to the heavy evaporation from a lake through which it flowed.

The Authors referred (p. 81) to the lack of information regarding

[1] C. S. Slichter, "The Motion of Underground Waters." U.S. Geol. Survey Water-Supply and Irrigation Paper No. 67 (1902).

—— "Theoretical Investigation of the Motion of Ground Waters." U.S. Geol. Survey, Nineteenth Annual Report (1897–98, Part II), p. 295.

e flows of rivers in Great Britain, and to the Commission proposed Mr. Roberts. ' the Water Power Resources Committee in their final Report of 21. He had been nominated by the Ministry of Health on that mmittee in 1919, together with Sir Frederick Willis and Mr. Sandrd Fawcett, C.B. Although he had not been appointed he had done considerable amount of work for the Committee and also, after it d reported, in the Ministry of Health in connexion with the survey domestic-water resources, which the Ministry afterwards made partmentally. It did not seem to him that in Great Britain the nefits to be obtained would justify the formation of such a Comssion as that proposed, or that a great expenditure of public money s called for in investigating the flows of a large number of rivers, cept for particular purposes, since, as Mr. Binnie had pointed out, eat Britain possessed excellent records of rainfall. Owing to the mplexity of the whole matter it would seem to be necessary, whater records existed of any general investigation, to make further vestigations of the flow of a river if any important works concerned th water were contemplated in its basin.

Mr. W. N. McClean observed that the late Mr. F. W. Macaulay, Mr. McClean. d Mr. Wilkinson and the Birmingham Corporation, had assured e maintenance of water-level records on the recorder up to the esent time, and that it was up to the Catchment Board to mainn those records and to extend the survey by some additional uging-stations, such as, for instance, at Ironbridge and at about miles west of Shrewsbury.

His own organization of " River Flow Records " had in 8 years complished over 300 similar gaugings on nine rivers from catchnts of from 85 square miles to 700 square miles, and twelve ter-level recorders were in use. Most of the differences in hnique between the Severn gaugings and his own were due to e organization of the work. His organization was for the purpose survey and in from 3 to 4 months the gaugings were complete d no stage of the river had been missed. The perfection of the paratus had allowed continuous gauging throughout floods. It s that continuity which allowed the correct water-level to be ed for each gauging vertical instead of an average level of about ours over the whole river.

His first gaugings had been on the river Garry in 1912–13. The ice of river had been fully justified. In 1929, at the request of Alexander Gibb, he had given evidence as to those flows before Committee which was considering the West Highland Water wer Scheme. From that date onwards, he had concentrated on flow-gaugings, water-level records and rainfall of the Ness basin d other areas in those parts of Scotland. For the recent Cale-

Mr. McClean donian Power Scheme the records of the Ness rivers had been available to all parties. It appeared likely that Inverness and the other water interests in the area would take over the maintenance of the

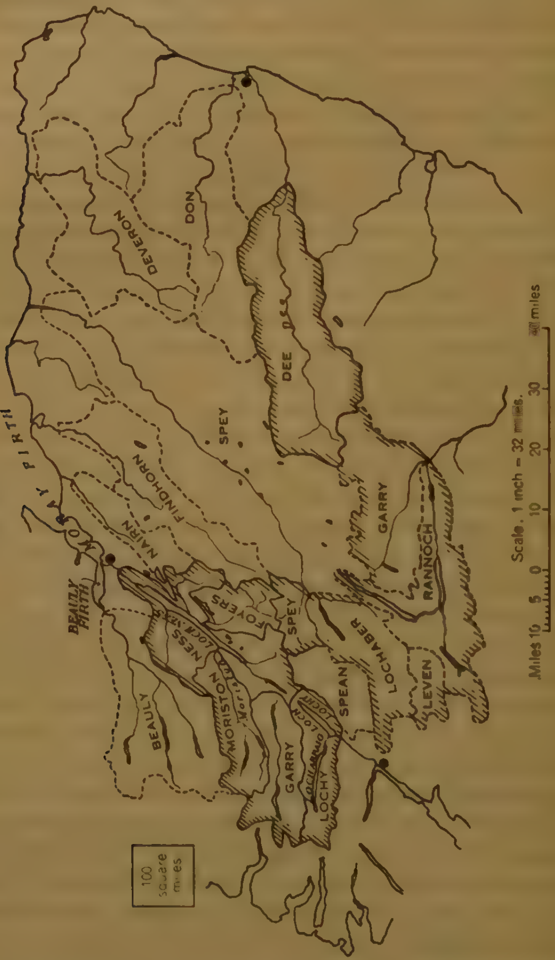


Fig. 26.

water-level stations. That was already done on the Dee, Spey and Foyers. The areas covered by his gaugings and records during the last 8 years, and by the large water-power schemes in the neighbourhood, were shown in *Fig. 26*. His apparatus, in several ways developed from that of Prof. Dixon's, was now to be installed on the Chester Dee, where the gaugings would be carried out by the staff of the Catchment Board after some preliminary instructions.

In every case the principal difficulty to overcome was access to e proper gauging and water-level sites. It was clear that river rvey required to be accomplished through the establishment of mparatively few survey stations, but they would have to be suit- le sites; further, inland water survey required the proper cilities. The maintenance of the records was bound to be depen- nt on the obligation of water interests to make observations and keep records. Mr. McClean.

When a straight reach of 300 yards was available and the bed of e river had been surveyed, there should certainly be full data as water-slopes, water-sections and mean velocities up- and down- ream of the gauging-section. They might not be necessary for ures of water resources but they were of great importance in annel-design. The two gauging-sites on the Severn were 2½ miles art and that was due to the straight reach being unsuitable for gh floods. It would appear that the reach was not too good for w flows, as the maximum velocity was only 0·75 foot per second d the mean velocity was very low indeed. A single gauging-site ould frequently be unsuitable for all stages. The important con- leration was that the site for the permanent water-level recorder ould be suitable for all stages and should not be affected by anges of bed. On his surveys, an extra gauge-post was frequently stalled to check the changes in river water-levels.

The coefficients used for floats (Appendix I, p. 110) were rived from the depth–velocity diagrams. On the current-meter ugings, mean velocity in a vertical was expressed as a coefficient surface-velocity. Surely, the coefficient applicable for each parate vertical had to be applied to the float-surface velocity of at vertical? The shape of depth–velocity diagrams appeared be governed not only by depth but by the acceleration or tardation at the section and on the reach. The analysis of that pended on slopes and water-sections above and below the gauging ction.

There was a great deal in the Paper about accuracy of measure- ents. The degree of agreement between gaugings was hardly a mplete measure of accuracy; nevertheless, on good reaches, it peared to be safe to say that gaugings might agree to better than per cent. The biggest change of level (0·3 foot) during gauging ould give from 100 to 300 cusecs difference in discharge. That peared to be 3 per cent. in itself. The averaging of water-levels ring gauging was a necessity of casual gauging but was indefensible accurate survey. It did not arise under his system, where the age–flow curve was made for each vertical; that course pre- nted no difficulty. The necessity for the proper method was obvious

Mr. McClean. from the hydrograph of June 1, 1924 (*Fig. 12*, p. 98) when, for 4 days, the rate of change was 0·6 foot per 6 hours' gauging time from 18-foot stage down to 10-foot stage. It so happened that on the actual Severn gauging it had produced very little error. The error would, however, be a very different matter on the upper reaches of the Severn. His method of taking readings was the same as the Authors', but his interval of 2 minutes for observation was certainly preferable to the 1 minute used by the Authors.

The formulas which had been hitherto put forward had been based on the relationship of mean velocity to water-section and wetted perimeter. It was very doubtful whether such a general relationship was going to be usefully applicable to most cases. Depths and the effects of bed-alignment on acceleration and retardation of flow would, in his opinion, outweigh any variation in coefficient of bed-friction due to variation in wetted perimeter.

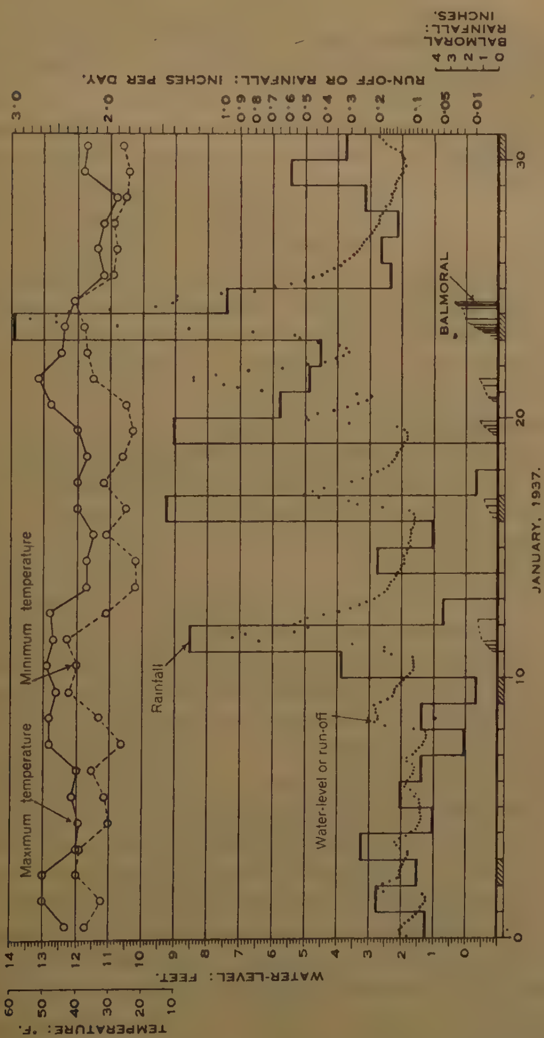
The Paper showed on p. 94 that the monthly quantities of run-off were not seriously affected, on the Severn at Bewdley, by the use of one daily reading. The reason for that was obvious from an analysis of the hydrograph of the flood of June, 1924. A midnight reading for each 24 hours would give 2,000 cusecs for June 1 instead of 12,000, but on the 4 days' fall that loss of 10,000 cusecs would be made up by an excess of 2,000 cusecs on each of the 4 days and on the continued drop.

On other rivers, when the peak was of short duration, there would be considerable error, as was shown on his hydrograph of the Dee flood in January, 1937 (*Fig. 27*). It might not matter much for statistics of water-resources, but the whole story of the passage of floods and of the storage and loss of several lengths of river was lost, and knowledge of those was of supreme importance in the control and uses of a river and for the prediction of floods. His records were diagrammatic, with an aggregate diagram of rainfall and run-off, showing also the storages of the area from day to day. It was his opinion that the Inland Water Survey Committee, in adopting tabular statistics relating to water-resources, had lost a great opportunity of placing the survey of British rivers on the right plane of British achievement.

The characteristic derived from the duration-curve was considered to have considerable stability over, say, a period of 10 years. He doubted if it had much stability on large rivers, and it would be of interest to know the annual variation between 1929–30 and 1930–31 on the Severn; the characteristic appeared to be 0·33 and 0·48 respectively. That hardly indicated a very stable value. It was generally considered to serve the purpose of Compensation

ater Assessment. Nevertheless, the true characteristic of a river Mr. McClean.
y in the effect of storages and losses, which were latent in the
te of rise and fall of a river; that was especially true in the case

Fig. 27.



FLOW OF RIVER DEE AT CAIRNTON GAUGING-STATION.

the fall, when the incidence of rainfall was eliminated from the
drograph.

The assessment of rainfall on a monthly basis from the area
hyetals of the month or year and the monthly isopercentals of the

Mr. McClean. several gauges could not remove the error which was unavoidable i reckoning the distribution direct from the daily reading of rainfal The daily assessment of rainfall should, he thought, be made, possible, on standard lines, and he adopted a method which, on larg areas, gave results very similar to the monthly method. Th comparison of rainfall and run-off could only be made at low-wate dates following a period of rain, when the balancing figure c residual run-off was known from the analysis of dry periods.

The losses due to the dry summer were very far from complete o the 1st October, and very large absorption-losses occurred i October and later. That was self-evident in the Severn table c losses, the 15 years' records for October amounting to 39 inches which was as great as for July. It was clear that annual losses reckoned on January 1 or a later date, when saturation was complete would always be more uniform in value. The accurate balance sheet of rainfall and run-off, for his records, was made during a dr period nearest to October 1, when the residual run-off could b calculated with considerable accuracy.

The rating diagram for a current-meter (*Fig. 20*, p. 112) showe how the curve below a velocity of 1·0 foot per second replaced th straight line of the higher velocities. It had been proved at th National Physical Laboratory and at the City and Guilds Colleg that at ½ foot per second his meters gave quite regular results; a present, however, the National Physical Laboratory tank was no equipped to rate intermediate low velocities. That difficulty ough certainly to be overcome at the earliest opportunity. For gaugin with a suspended current-meter it seemed clear that the Ott loade current-meter ought to replace the sinker-weight beneath the meter.

There ought to be no doubt that engineers in Great Britain required records of water-level, flow and run-off, and storage.

The drought in 1933 had demonstrated the importance of water resources, and possibly the intense floods of the 1936–37 winter in Scotland, in the west and in the Fens, would encourage surveys. The problem of the Fens and of many estuarial areas was that o storage. The problem of any river-flood was largely concerned with the concentration-time and its dependence on storage. Th December, 1936, flood on the Lochy, Garry and Moriston, all with a catchment of 150 square miles, was 6,000, 10,000, and 16,000 cusecs respectively with very similar rainfalls. Those peak values were, respectively, equivalent to 1½ inch, 2½ inches and 4 inches per day intensity, and the variations were entirely the result o Loch storage. He estimated the concentration-time at from 2½ to 3 days for the three hydrographs in the Paper. If, on June 1, 1924 the rainfall of 3 inches were spread over 3 days, in winter time

ere would have been a steady rise over those 3 days giving a peak- **Mr. McClean.**
ensity rate of 1 inch per day, or 44,000 cusecs. On the Dee, that
me rainfall in January, with a concentration time of 24 hours,
ve a peak at the rate of 3 inches per day from an area of 528 square
les.

Mr. C. F. LAPWORTH desired to deal quite shortly with the relation **Mr. Lapworth.**
tween storage and yield of the particular catchment. The present
cords included the dry period from November, 1932, onwards,
d it had been during that period that the resources of the various
tchments over the greater part of the country had been taxed
ore severely than at any time since 1887. A comparison with the
perience in other catchments was, therefore, interesting.

For a number of catchments where the data was available he
d plotted the monthly run-off figures in the form of a cumulative
agram and had evaluated the storages required to give various
elds. Those results had been plotted as a curve, one for each
tchment. From those curves he had derived a formula which
ve the relationship between storage and yield. The formula was
adaptation of one put forward jointly by his father and Mr. Binnie
1913. That formula had originally been "put forward tenta-
ely and as a temporary expedient of an empirical nature." The
mula had been largely based on run-off records then available,
vering the dry period 1887–88. A study of the run-off records
eady referred to indicated that the formula needed revision in
e light of the recent drought. The present formula, which was
ain suggested tentatively and as a basis subject to revision, was
ended to give sufficient storage to cover requirements of catch-
nts where the 1933–34 drought had been most severe—conditions
ich might be expected perhaps once a century. The revised
mula was $S = \dfrac{8(y-d)^2}{(Y_{max})^{1 \cdot 6}}$

ere S denoted the effective storage-capacity of the reservoir expressed in inches over the drainage-area,

y ,, yield required in inches per annum,

d ,, minimum dry-weather flow over 1 month in inches per annum,

Y_{max} ,, maximum yield in inches per annum, or the long average rainfall minus the average loss.

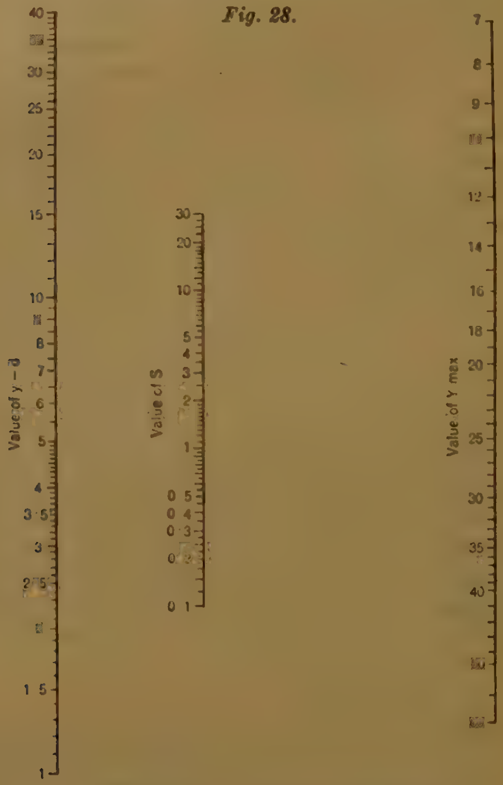
nstants :—

Inches per annum $= \dfrac{\text{Gallons per day}}{(\text{Area of Gathering Ground in acres}) \times 62}$.

Storage in gallons $= S \times$ (Area of Gathering Ground in acres) $\times$ 22,610.

Mr. Lapworth. The formula appeared to give reasonably good results for yiel up to 80 per cent. of the maximum yield, and for catchments wit rainfalls varying between 26 and 75 inches. Advantage had bee taken of the logarithmic nature of the formula to prepare an alig ment chart (shown in *Fig. 28*), which might be useful in the pre liminary design of storage reservoirs by making assumptions i

Fig. 28.



Range of data. Y max. = 8 to 60 inches per annum
Range of formula. Y up to 80 per cent. maximum yield

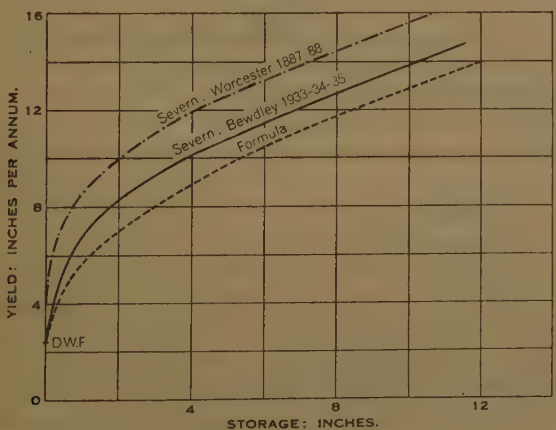
respect of average run-off and dry-weather flow, and later in con junction with actual gaugings, which might not include a very dr period, in determining the final design.

In the case of the Severn, the average run-off for the period c gauging had been 18·9 inches. During that period, however, th rainfall had been 110 per cent. of the long-period average; makin an adjustment for that, the long-period average run-off woul appear to be approximately 16·6 inches per annum. The dr

eather flow was taken as 2·2 inches per annum, which was the onthly flow in September, 1933. Mr. Lapworth.

Fig. 29 was a yield–storage diagram for the Severn. The full line dicated the storage required for the period 1933–35, taken from e figures given in the Paper. A comparison was shown with the orage required in the 1887–88 period taken from gaugings at orcester and referred to in the Paper. It would be seen that the cent drought had required larger storage throughout the range. ne lower dotted line indicated the results given by the formula. comparing that curve with the curve of actual storage required uring the 1933–35 period it would be seen that, although the neral form was similar, the storage given by the formula was mewhat greater than that actually required. For a yield equal three-quarters of the average run-off, the difference was about per cent. The discrepancy was accounted for by the intensity the drought in the Severn valley.

Fig. 29.



The period of twenty-five months from November, 1932, to vember, 1934, had given the largest deficiency over England and ales as a whole. The Severn catchment down to Bewdley vered an area over which the rainfall had been approximately) per cent. of the average annual rainfall. There had been, how- er, an area in the South Midlands, and another in South Wales, ere the rainfall had been less than 150 per cent. of the average. e figure for the Thames valley had been 144 per cent. of the erage, and in that case the agreement with the formula was very se.

Mr. Lapworth. The difference in storage between that given by the formula and that actually required represented a margin against the possible recurrence of a drought of similar intensity to that experienced in other areas.

Mr. Wileman. Mr. R. F. Wileman said that it would be of interest to know if the velocity-depth curves had been analysed with a view to confirming the reliability of the "mid-depth" method for determining the mean velocity. He believed that that method was in use in many parts of the world, as were other methods involving the determination of mean velocity by the observation of velocity at one or two points only in the vertical. It would be of interest to see if the factor for reduction of mid-depth velocity to mean velocity used for large open water-channels in other parts of the world, was confirmed by the Severn measurements. That factor, which had been adopted in Egypt, was 0·96, and had been checked in Egypt by a very large number of observations of velocity-depth curves. He had been told by Dr. Percy Phillips of the Physical Department of the Public Works Ministry of Egypt that it had been found that the ratio of the velocity at mid-depth to the mean velocity in the vertical was less changed by special conditions, such as strong winds with or against the stream, than the ratio of the velocity at any other single point to the mean velocity.

The saving of time and the obvious simplicity of operation resulting from velocity-observation at mid-depth only, was considerable not only in the field but in computation, and if, as seemed likely, many discharge-measurements were required in Great Britain that aspect of the question would be of importance.

Mr. Gold. *** Mr. E. Gold considered that the hydrographs shown in *Figs. 7* to *11* (pp. 96 and 97) provided not only an interesting picture of the years' events in the Severn, but also a basis for a study of the relations between flow of the river and the meteorological factors of rainfall, wind, humidity and sunshine. Were the daily discharges plotted to form the hydrographs derived from single daily readings? It was clear from the results given earlier in the Paper that single daily readings formed a good basis for means or averages, but they were not likely to be accurate representations of the flow on individual days. The fact that other factors besides rainfall were bound to play a part was evident from an examination of Tables I and II (pp. 102 and 104). If, from those two Tables, the ratio of the run-off to the rainfall were computed for each year, the

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*** This and the succeeding contributions were submitted in writing.—Sec. Inst. C.E.

lowing were the results if the ratio were expressed as a Mr. Gold.
rcentage :—

Year.	Run-off/Rainfall per cent.
1921–22	44
1922–23	52
1923–24	51
1924–25	51
1925–26	43
1926–27	46
1927–28	61
1928–29	46
1929–30	52
1930–31	56
1931–32	51
1932–33	53
1933–34	33
1934–35	42
1935–36	54
Average	49·5

e smallest ratio occurred in the driest year ; but the greatest ratio
l not occur in the wettest year: in fact it occurred in a year,
27–28, with rainfall just below the average.

It was clear that the ratio of run-off to rainfall increased, broadly
aking, as the rainfall increased. In fact, if those percentages were
otted against the rainfall, they grouped themselves broadly about
straight line, from which the following figures were derived :—

Rainfall: inches.	Ratio of run-off to Rainfall: per cent.
20	30
30	40
40	50
50	60

e most exceptional years of the series were :—

Year	Ratio: per cent.	Rainfall: inches.	Computed ratio: per cent.
1927–28	61	38	48
1926–27	46	45	55
1932–33	53	29	39

The observed ratios therefore differed by + 13 per cent., — 9 per
t., and + 14 per cent. from what might have been expected from
rainfall totals on the basis of the results from other years. What
s the character of those 3 years which determined the large
parture from the average in that ratio of run-off to rainfall ?

Mr. Gold. He thought that it was only by a careful examination of the individual days or individual months in the exceptional years that the causes of the variations of that ratio would be found. The departures were much too great to be due to accidental errors, and he considered that they were bound to have been associated with exceptional conditions of wind or sunshine in the catchment-area. Figures of mean velocity of wind or mean sunshine for the year were not likely to be of much help; it was the effect of wind and sunshine at the time when the ground was wet or immediately following rain which would have the predominating influence.

Mr. Gourley. Mr. H. J. F. Gourley observed that the Authors drew attention on p. 90 to the form of the curve for the mean hydraulic radius, as shown in *Fig. 4*. That radius, according to that curve, reached its maximum value of 12·55 feet at gauge-height at 17·0 feet; that was to say, at a discharge of 12,000 cusecs. The Authors observed that the subsequent reduction in the hydraulic radius was a consequence of the river then overflowing its banks. He would point out that the shallow marginal flow was relatively small and often negligible, for observation in such cases indicated that the velocity of flow in the margins was quite small as compared with that of the main channel; he suggested that it was not correct to treat the section as one channel, but rather that it should be treated in two portions, the one being regarded as having an area and a wetted perimeter obtained by extending verticals from the sides of the channel to the water-surface, and the other being taken as the balance of the section.

The form of the discharge-curve (*Fig. 5*, p. 91) rather suggested that no credit had been taken for the flow over the marginal lands, and in any case a reference to the average duration-curve (*Fig. 15*, p. 100) showed that flows exceeding 12,000 cusecs had only occurred during about $1\frac{1}{2}$ per cent. of the time, or on about 70 or 80 days in 15 years, so that no matter how the marginal flows had been dealt with the error in the record could not be material. So far as could be ascertained from the average duration-curves the Severn discharged at or below its average during about 66 per cent. of the time. The Thames at Teddington had a drainage-area of 3,812 square miles and the percentage in that case was 65 per cent. It was found that over a period of years the flow of British rivers was equal to, or less than, the average during from 65 to 70 per cent. of the time.

The annual rainfall and run-off figures from Tables I and II, which were plotted in *Fig. 17* (p. 107), were approximately represented by the equation $F = \frac{2}{3}(R - 10)$ in which F denoted the annual run-off in inches and R denoted the corresponding rainfall in inches. In individual years the difference between the calculated and the

ual run-off ranged from about $\frac{1}{2}$ inch to $2\frac{1}{2}$ inches; the average Mr. Gourley.
culated run-off of the 15 years was, however, 18·90 inches, and
t happened also to be the actual average run-off.

During the 3 years 1932–35 the total rainfall was 89·13 inches, l those were the lowest three consecutive years, not only in the ole but probably for 60 or 70 years. The corresponding run-off s 38·07 inches and the average annual loss during those years s 17·02 inches, as compared with the average over the 15-year iod of 19·24 inches. It might seem obvious that if the rainfall re less the opportunities for loss were less, but that fact was somees overlooked, and the figures just mentioned proved the fact.

Converting the formula just given to give the loss L, the result s $L = \frac{1}{3}R + 6\frac{2}{3}$, and that again showed that the loss had two nponents, one a constant and the other which varied approxitely with the rainfall.

Mr. Ben Howorth observed that the choice of Bewdley for a Mr. Howorth.
ging-station for the river Severn appeared to have been made ng to the fact that it was on a length of river stated to be free m artificial controls and river-traffic. Such ideal conditions e always very difficult to find, and on rivers like the Lee and ames were almost impossible to obtain. The river Lee and its utaries had either locks or mills, or both, situated at short ervals from one end to the other. The problem in that case was easily solved by the establishment of a gauging-station of the e described in the Paper. A further difficulty occurred in the nge in condition of the channel, the shape of which might alter y considerably during times of flood. In the case of Bewdley it stated that a prominent ledge of rock ran obliquely across the r near the station, and that undoubtedly helped to maintain a form section. A similar condition would be hard to find in most rs.

The Catchment Boards were in most cases establishing gaugingions on their rivers and it was clear that the information in ard to the height of floods and their duration was the factor of st importance to such Boards. For that reason the records had be continuous, and that meant the installation of automatic ording instruments. Further, those instruments ought to be brated to read directly the quantity of water passing the station, ch meant that the stations had to be correctly designed. Weirs, turi flumes, or channels of uniform sections would be necessary that reason, and such works entailed heavy expenditure in the instance. Automatic diagrams to show quantities saved a ge amount of labour in calculation, and as very few Catchment ards could find the staff necessary to make extensive calculations

Mr. Howorth. that was a very big consideration in the choice of the method to be adopted in the gauging of a river.

The calculation of the run-off and its relation to rainfall which the Authors made was of great interest at the present time, and if the various catchment-areas were in any way similar to the Severn and to each other the average discharge for the various rivers could be easily found owing to the widespread activities over long periods of the British Rainfall Organization. As it was, every catchment-area had characteristics differing from all other areas, and the only satisfactory way to obtain the run-off was to measure it by means of as many gauging-stations as the particular area required. Where such stations were established the particular relationship of run-off to rainfall would, however, lose its interest, as the run-off would then be a known quantity instead of, as at present, a matter of estimation based on rainfall.

Mr. Lacey. Mr. J. M. Lacey pointed out that the land of the catchment-area supplying the water might have no intrinsic value, but when assessed in terms of the water it produced, and of the human welfare dependent on its supply, its value was bound to be considerable. It might be expected that the source of such wealth would be given some attention, but in actual practice little had been done to preserve its value as a useful water-producer. In late years, particularly in America, attention had been directed in the study of flood-control to soil-preservation and to the control of "little waters," so that in addition to actually measuring the quantity of water available from a catchment-basin, it seemed equally important to consider how best its value as a water-producer could be conserved.

Where there were durable and resistant beds and banks capable of maintaining the channel-section with little change from year to year, and where the bed-slope was such that slight variation in surface-slope did not materially affect the discharge, the stage-discharge relation was reliable. Where the river-channel was not so nearly permanent, however, and particularly where the channel was notably unstable, the records of stages were less reliable if converted into corresponding discharges. The tendency of rivers to erode, and to deposit bed-loads in the vicinity of the gauges materially affected the relation. The stage–discharge relation was also affected by phase of flood, and by the manner of successive flood-rises. In some cases two curves were given, one for discharge at any stage "rising river," and one for discharge at any stage "falling river." Even with that provision discrepancies arose, as with a falling flood alluvial rivers generally scoured their beds further, although, for the same stage, the mean velocity of a rising river might be greater than that of the falling river, the increased

ea of water-section at the latter stage might give an equal and Mr. Lacey.
rhaps a greater discharge. As an example, Table X, which was tracted from the Report of the Indus River Commission, gave e hydraulic details of the river Indus at Sukkur, one of the uging stations, for the flood season of 1914. Observations for scharge were made at the Outfall gauge. The mean depth was e area of the water-section divided by the width at the water-

TABLE X.

Date 1914.	Goa Ghat gauge.	Outfall gauge.	Fall in 3,642 feet.	Mean depth.	Mean below datum.	Mean velocity.	Discharge: cusecs.
ne 27	10·5	$10·1_r$	0·41	14·52	18·92	7·34	363,666
ly 1	12·0	$11·2_r$	0·81	17·30	20·42	7·14	417,153
,, 4	12·6	$11·7_r$	0·91	18·12	20·92	7·44	459,823
,, 8	13·7	$12·6_r$	1·11	20·80	22·70	9·46	671,876
,, 11	13·8	$12·8_r$	1·01	21·83	23·58	9·58	715,346
,, 22	13·0	$12·3_f$	0·71	22·65	24·85	9·51	734,439
,, 25	12·9	$12·3_{oo}$	0·61	22·58	24·78	9·24	710,931
,, 29	14	$13·0_r$	1·01	24·29	25·79	9·68	802·758
ıg. 1	14·7	$13·7_r$	1·01	27·19	27·99	10·18	948,928
,, 4	Maximum Flood.—No hydraulic details. Estimated 14·4 on outfall gauge.						
,, 5	15	$14·1_f$	0·91	28·76	29·16	8·96	885,165
,, 8	13	$12·6_f$	0·41	30·90	32·8	6·94	710,656
,, 12	10·5	$10·2_f$	0·31	26·85	31·15	4·43	405,352
,, 15	12·9	$12·3_r$	0·61	27·34	29·54	6·35	593,156
,, 19	11·2	$10·7_f$	0·51	24·90	28·70	6·31	534,660
,, 22	10·0	$9·5_f$	0·51	21·72	26·72	6·19	457,266
,, 26	—	$9·9_r$	—	20·29	24·89	7·29	502,799
,, 29	9·6	$9·1_f$	0·51	19·00	24·40	6·68	429,866

ırface, and the mean below datum was the mean bed of the river elow a fixed datum-line, and served to show the changes which ccurred in the bed of the river. The datum to which all areas were ferred to was fixed at 14·5 feet on the Outfall gauge. The subcript *r* on the outfall-gauge readings denoted a rising river during ıe past 24 hours, the subscript *f* denoted a falling river, and the ıbscript *oo* denoted a stationary river. The reduced levels of the eros of the Goa Ghat gauge and of the Outfall gauge were 184·14 and 84·13.

The procedure adopted by the Authors could hardly economically e applied to gauging a large river, apart from the possibility of the iver-level and slope altering during the time occupied in gauging s discharge. On the Indus river observations had been taken at very 100 feet of the cross-section from a launch kept under steam gainst the current, its position having been fixed by cross-bearings.

Mr. Lacey. The mean velocity at each vertical section, from numerous vertic velocity observations, was assumed to be at six-tenths of the dept That single observation had given a satisfactory mean velocity the vertical section. The true mean, for that river, of sever thousand vertical-velocity observations taken from 1901 to 19 showed the true mean in a vertical to be at about 0·57 of the dep The maximum velocity had been found at the surface in most case

The Indus River Commission observations from 1901 to 1923 ga the ratio of the mean velocity to the velocities at various depths follows :—

Surface.	$\frac{1}{10}$	$\frac{2}{10}$	$\frac{3}{10}$	$\frac{4}{10}$	$\frac{1}{2}$	$\frac{6}{10}$	$\frac{7}{10}$	$\frac{8}{10}$	$\frac{9}{10}$
0·843	0·874	0·896	0·921	0·944	0·973	1·010	1·062	1·128	1·22

The bottom velocity could not be measured with the meter. Tho figures were the mean, as stated above, of several thousand observ tions in that river. A detail examination of the records of th vertical velocities given in the Commission's Report showed th curve of vertical velocities to vary for certain conditions of th river. In a rising flood there seemed to be little difference betwe the velocities at the various depths, and in that connexion attentio might be drawn to Professor Unwin's remarks on a Paper presente to The Institution.[1] The relation between the mean velocity in vertical and the maximum velocity in that vertical depended als to a certain extent on the resistance of the bed and sides of th channel. The amount of silt transported also seemed to have som effect on the ratio of the mean velocity in a vertical to the maximun velocity in that vertical. The relation of surface-slope to bed-slop in a channel also influenced the curve of vertical velocities, th curve becoming more vertical as the surface slope became greate than the bed-slope. In regard to the latter statement, had th Authors any records of vertical-velocity observations taken whe the surface-slope of the stream was greater than the bed-slope ?

The Authors' method of gauging the river was a council of perfection, but a fairly practical estimate of the discharge of a stream could be made by adopting Harlacher's method of gauging by mean of surface-floats.[2] The method was simple, no current-meters with their associated equipment being required, but although it was so

[1] W. R. Browne, "The Relative Value of Tidal and Upland Waters in Maintaining Rivers, Estuaries, and Harbours." Minutes of Proceedings Inst. C.E., vol. lxvi (1880–81, Part IV), pp. 38 and 39.

[2] Irrigation Pocket Book, 1911 edition, p. 160. London.

nple, it was never used. It was essential, however, that the Mr. Lacey. rface-slope of the stream should be obtained by means of a level, from two gauges set some distance apart in the reach being uged, and that it should be recorded at the time of gauging.

The relation between rainfall and run-off or the surface-yield of a drographic basin had been dealt with elsewhere.[1]

Mr. F. O. STANFORD drew attention to an earlier Paper [2] on the Mr. Stanford. bject of river discharge, published by The Institution nearly fifty ars ago. The description on p. 85 and *Fig. 2* (p. 86) of the Paper der discussion bore such a striking resemblance to what was own as the Harlacher method that a reference to the original per would seem appropriate. It was of interest to note, o, that the float-coefficient of 0·85 mentioned on p. 92 was ntical with the figure for the proportion of mean velocity to rface-velocity arrived at by Harlacher.

Mr. R. C. S. WALTERS observed that the period 1921–1936 covered Mr. Walters. o remarkable droughts and probably some periods of very high ods, and for engineers concerned with waterworks data such as ose given in the Paper were most valuable.

There were three points of outstanding importance to be studied river-flow records :—

(i) The dry-weather flow during a period such as 1 month, which was probably more instructive than the smallest flow in any one day. That figure was used directly when it was desired to abstract a portion of the river-flow without resorting to storage.

(ii) The maximum flood occurring during such a period as 1 day. That was directly used for the purpose of economically designing the overflow-weir of an impounding reservoir. In most cases the reservoir could take care of a peak flood during 1 hour, but not of a peak flood lasting 1 day or more.

(iii) The way in which the flow occurred. That was of direct fundamental importance in determining the relationship between storage and yield.

Reliable data were very scanty in Great Britain for such a large ver as the Severn. He noticed that at Bewdley the lowest flow ring the month amounted to 0·202 cusec per thousand acres, sed on the flow of July, 1921, or 0·192 cusec per thousand acres, sed on the single day's flow in July, 1921, referred to on p. 101.

[1] J. M. Lacey, "Hydrology and Ground Water." London, 1926.
[2] A. R. Harlacher and H. Richter, "A Simple Method of Ascertaining the scharge of Rivers." Minutes of Proceedings Inst. C.E., vol. xci (1887–88, rt I), p. 397.

Mr. Walters. It was satisfactory to have the very common allowance of $\frac{1}{5}$ cus per thousand acres confirmed in that way. Incidentally, the corr sponding figure for the Thames at Teddington during July, 193 appeared to be 0·187 cusec per thousand acres, falling to 0·11 cus per thousand acres on the 9th July, 1934. A gauge had recent been established on the Mississippi at Vicksburg which command an area of no less than 732,480,000 acres, and it was interesting observe that the lowest dry-weather flow during the period of Jul 1931, to September, 1934, worked out at 0·225 cusec per thousa acres in November, 1931, or 0·173 cusec per thousand acres on th 19th November, 1932. It appeared, therefore, that the commonl accepted allowance of $\frac{1}{5}$ cusec per thousand acres was substantial correct for large gathering grounds.

With regard to floods, he observed that the greatest flood recorde in a day in January, 1925, was 18,200 cusecs, or 17·2 cusecs pe thousand acres. That was very much lower than a flood from an upland area such as might be estimated from a prolongation of th curve given by the Floods Committee. Similarly, the figure for th Thames flood of 1894 was 15 cusecs per thousand acres and that fo the Mississippi flood of the 26th February, 1932, was 19·3 cusecs pe thousand acres. On p. 101 the Authors had given the ratio betwee the lowest daily discharge and the highest daily discharge of th Severn as 91. That ratio was 137 for the Thames and 113 for th Mississippi.

The Authors. The AUTHORS, in reply, observed that the Discussion had amplifie many of the topics touched on in the Paper and that they wer indebted to the contributors for having given such full details o river-gauging operations elsewhere. The remarks of Mr. Binnie o the need for hydrometric work, and of Sir Clement Hindley on th action which was now being taken to meet that need, were of especia interest, and it was satisfactory to learn that the Inland Wate Survey Committee were about to commence publication of a Yea Book.

During the 15 years covered by the records the discharge exceede 10,000 cusecs on 62 days. In the four years 1921–22, 1932–33 1933–34, and 1934–35 the discharge did not exceed 10,000 cusecs The frequency of the different flood-discharges during the 15 years was :

Discharge in excess of	Days.
18,000 cusecs	1
17,000 ,,	2
16,000 ,,	6
15,000 ,,	12
14,000 ,,	20
12,000 ,,	33

studying the ratios of rainfall and run-off for the Severn and The Authors.
ames, it should be remembered that the period covered by the ern records was too short to yield a reliable average for the ividual months. Records of the flow of the Thames were avail-e for a much longer period, and when the Severn records were ended, a closer agreement might be found to exist.

he Authors were unable to follow Mr. Allard when he suggested t the Paper dealt with work in deep swift water. The equipment river-gauging had to be proportioned to suit the natural con-ons, and at Bewdley those were favourable, the maximum depth l velocity being about 16 feet and 7 feet per second respectively. weight of the sinker was obviously bound to depend on the depth l velocity of the stream, and on the drag of the suspension-cable l meter; a 30-lb. sinker was adequate at Bewdley. At an early ge in the investigation it had been found that the heavy cab-tire le supplied with the Price meter gave rise to excessive drag. A vanized stranded-steel cable 1/8 inch in diameter gave much better ılts. The steel cable was connected to the meter through an ılated joint and about 2 feet of hemp cord. Unlike the arrange-nt advocated by Mr. Allard, there was no insulated lead on the e, and the steel cable was used as the positive lead of the electrical uit through the meter, a short length of insulated flexible wire ning the end of the cable to the meter contact-box. The return uit was from the meter-body through the water to an earth-plate he bank.

t was not suggested that there was any novelty in the cable-pension method of gauging—a somewhat similar apparatus had n used by the late Dr. G. F. Deacon [1] in 1881—but the Authors that the advantages of that method had been rather overlooked. Ott suspension gear might have certain advantages where the th of the river was sufficient to render communication between the banks difficult, but the concentration of the controls at one point ıld be likely to increase the difficulties of erection. The gear used Bewdley was designed to be light, portable and easily erected, ause under normal circumstances the engineers of a Catchment rd or similar Authority would wish to maintain a number of ging-stations and to carry the apparatus around from station to ion.

he "paravane" device (*Fig. 22*, p. 120) for causing a meter to was interesting, but in the absence of experience with that type

H. T. Turner, "The Gauging of Flowing Water." Minutes of Proceedings . C.E., vol. lxxx (1884–85, Part II), p. 327.

The Authors.

of meter the Authors feared that errors might arise due to tl flutter of the paravane.

Mr. Hibbert's close analysis of the floods of May–June, 1924, an February, 1928, in the light of the Vyrnwy records showed ho valuable information could be obtained from a comparison of di charge-records at different parts of a catchment. The estimate 52 per cent. loss was arrived at from a study of the hydrograph f the flood, but in the Authors' view the monthly totals also confirme that estimate. The rain in May and June, 1924, was 3·31 inch above the average, and the discharge for those 2 months was 1·69 inc above the average, the excess loss being 1·62 inch or 49 per cent. . the excess rainfall. No trouble had been experienced with ice durin the period of the records.

The detailed account of the gear devised by Messrs. McClean an Chapman for use on Scottish rivers again emphasized the importanc of adapting the equipment and methods to the character of the rive under observation. Where expense was of importance, less elaborate arrangements might have to be used, but the Authors' views were intermediate between those of Mr. McClean, who advocated 2-minute observations, and 6 hours' gauging time, and those of Mr. Wileman who favoured a single observation at mid-depth on each vertica Under conditions of steady discharge the velocity of flow through th gauging-section varied with both time and position. At any on point there were velocity pulses, but if observations were continue for 1 minute the velocity obtained did not differ (at Bewdley) mor than about 4 per cent. from the average of a long period. Th velocity varied along each vertical and across the section, so that th problem of discharge-measurement was similar to that of finding th volume of an irregular solid with a rough or indefinite surface restin on a plane base. Practical considerations set a limit to the time which could be spent on a discharge-measurement, and a decisio had to be reached as to whether there should be a small number o very accurate velocity-observations, or a large number of more rapi and less precise measurements. Returning to the analogy of th irregular solid, it was obvious that a better result would be obtaine by taking a large number of moderately accurate spot-levels properl distributed over the surface than by taking a relatively small numbe of precise levels. Similarly, in river-gauging the Authors considere that in view of the pulsations in velocity and the variations in velocity-distribution, it was desirable to measure the velocity at a large number of points distributed over the cross-section in such a way that observations were most plentiful in the regions of maximum discharge. The number of observations naturally varied with the stage of the river, and at high stages the observations were not taken

every foot of depth. As a rule a measurement involved meter- The Authors.
servations at from one hundred to one hundred and twenty points
l occupied up to 2½ hours. If the rate of change of surface-level
re 0·1 foot per hour and the mean gauge-reading were taken, the
t and last observations would be 0·12 foot away from the mean.
a medium flow of, say, 5,000 cusecs, a change in level of 0·1 foot
the gauging section corresponded to a change of 100 cusecs dis-
rge, but that did not imply that the measurement would be in
or by 120 cusecs. Actually, the stage–discharge relation was
ctically linear in so far as small changes of level were concerned,
l the error involved in taking the measured discharge as corre-
nding to average gauge-reading would be insignificant when it
s remembered that the unit discharges for each vertical were
rected to their mean water-level value before the discharges were
culated.

The minimum discharge-measurement (210 cusecs) was taken at a
nt about 100 yds. above the gauging-section, where the low-water
nnel was shallower. The velocities measured on that occasion
ged from 0·48 to 1·40 foot per second, and the average velocity
the whole cross-section was 0·60 foot per second.

Mr. Bilham's statistical investigation of the rainfall–run-off
ation brought out the close correlation between them, and his
gestion that that might be used for the study of earlier run-offs
uld be further studied. Mr. Gold attacked the problem in a
erent way and inquired why certain years showed large deviations
m the average straight-line reaction. Both 1927–28 and 1932–33
re characterized by comparatively dry summers; most of the rain
in winter when the loss was small. In 1926–27, on the other
d, the rain was more uniformly distributed throughout the year,
l there were several falls in June, August, and September with
h losses.

The formula for storage-capacity quoted by Mr. Lapworth might
e satisfactory results within the range of the data from which it
l been derived, but since the formula was not dimensionally
nogeneous, exterpolation should be avoided. In calculating the
od-discharge due allowance had been made for the marginal flow,
Mr. Gourley's suggestion with regard to splitting up the cross-
tion when calculating the hydraulic radius deserved fuller investi-
ion. The reduction in hydraulic radius when the gauge-reading
eeded about 17 feet seemed to be connected with the reduction
slope between Elan aqueduct and Bewdley at high flows shown in
. *6* (p. 95). The slope might fall because the channel at the lower
l became less efficient due to the drop in hydraulic radius, but the
truction caused by Bewdley bridge probably had a similar effect.

The Authors. The surface-float observations were subsidiary to the main investi-gation and the results obtained were more consistent than th Authors had expected from a study of previous work on the subjec Owing to the comparative irregularity of the river-bed no form sub-surface or rod-floats could be used, and favourable conditio for those devices were of rare occurrence in Britain.

*** The Correspondence on the foregoing Paper will be publishe in the Institution Journal for October, 1937.—Sec. Inst. C.E.

EXTRA MEETING.

15 April, 1937.

Sir ALEXANDER GIBB, G.B.E., C.B., F.R.S., President, in the Chair.

Special Lecture on

"The Boulder Dam."

By John Lucian Savage, Chief Designing Engineer, Bureau of Reclamation, Department of the Interior, United States.

TABLE OF CONTENTS.

Boulder dam is located on the Colorado river about 30 miles south-east of Las Vegas, Nevada, at a point where the river forms the boundary between the States of Nevada and Arizona. The purposes for which the dam was constructed include navigation, flood-control, irrigation, domestic water-supply, power-development, and silt-storage. A general map is shown in *Fig. 1* (p. 162).

Chronological History.

The principal steps in the history of the project may be listed chronologically as follows :—

In 1902 President Theodore Roosevelt signed the Reclamation Act. From 1902 to 1918 the Reclamation Service investigated and reported on many projects for the control and utilization of the Colorado river. In 1918 Arthur P. Davis conceived a dam of unprecedented height in Boulder or Black Canyon. In 1922 the Colorado River Compact was signed in Santa Fe, New Mexico, on

Fig. 1.

Map of District.

the 24th November, by the Governors of the States of California, Colorado, Nevada, New Mexico, Utah, and Wyoming.

In 1924 the Reclamation Bureau reported on the Colorado river, recommending construction of the Boulder Canyon project. During 1928 the Boulder Canyon Project Act was passed by the Senate on

the 14th December, by the House on the 18th December, and the Act was signed by President Calvin Coolidge on the 21st December.

In 1930 contracts for the sale of electrical energy were entered into with the Metropolitan Water District of Southern California, the City of Los Angeles, and the Southern California Edison Company, on the 26th April. During 1931 bids were opened for the construction of Boulder dam and power plant on the 4th March, the contract was awarded to Six Companies, Inc., on the 11th March, and the work started on that same day.

In 1932 the diversion of the Colorado river was completed on the 14th November. In 1933 the first concrete was placed in the dam on the 6th June. During 1935 the storage of water commenced on the 1st February, the last concrete was placed in the dam on the 29th May, and the project was dedicated by President Franklin Roosevelt on the 30th September. In 1936 the regular delivery of electric power to the City of Los Angeles started on the 26th October.

Preparatory Features.

As soon as funds became available for the construction of the project on the 3rd July, 1930, every effort was made to place the work under contract promptly. Some of the more important preparatory features that were contracted during the early months include the following :—the Government portion of Boulder City and appurtenant facilities ; a 33-mile branch railroad from Bracken, Nevada, and a 32-mile oiled highway from Las Vegas, Nevada, to Boulder City and thence to the dam-site ; a 222-mile, 88,000-volt transmission line from San Bernardino, California, to the dam-site ; a 15,000-kilowatt substation at the dam-site, with branch transmission-lines to different parts of the project and to Boulder City ; a 150-ton cableway with a span length of 1,256 feet. All of these preparatory features were rushed to completion as fast as possible.

Six Companies' Plant and Equipment.

The Six Companies' construction facilities included the following major items :—the construction camp at Boulder City ; 45 miles of standard-gauge railroad complete with rolling stock ; 20 miles of construction roads ; one hundred and forty-five trucks of varying capacities up to 16 cubic yards ; twelve large electric shovels and draglines ; nine heavy-duty bulldozers ; thirteen tractors ; three air-compressor plants with a combined capacity of 18,000 cubic feet of air per minute ; a concrete-aggregate processing plant with a capacity of 20,000 tons per 24 hours ; a cement-blending plant with storage capacity for 48,000 barrels of unblended cement ; two

concrete-mixing plants with combined capacity of nearly 15,000 cubic yards of concrete in 24 hours; nine construction cableways of from 10 to 20 tons' capacity, with spans varying from 1,360 to 2,575 feet; three derricks of from 12 to 20 tons' capacity, with booms varying from 140 to 185 feet in length; a 3,000-gallon-per-minute water-cooling system and an 825-ton refrigeration plant for cooling mass-concrete in the dam; three large shops; two large garages; two warehouses; miscellaneous equipment and innumerable pumps, air drills, and tools of all kinds. The investment in construction facilities was approximately $7,200,000 and it is estimated that an expenditure of fully $2,000,000 was made before the first monthly estimate was received by the contractor.

Aggregate and Mixing Plants.

The concrete-aggregate and mixing plants were obviously the most important items in the above list of construction plant, and it seems pertinent to describe these briefly. The aggregates were obtained from the Arizona side of the river about 7 miles upstream from the dam. The gravel was composed of limestone, granite, basalt, and quartzite. The sand was almost entirely quartz and contained an excess of particles passing the No. 28 and retained on the No. 48 standard screens.

An electrically-operated aggregate plant was built on the Nevada side of the river about 5 miles upstream from the dam. The plant produced sand and four sizes of gravel at a maximum rate of 20,000 tons per 24 hours. The pit-run material was screened, washed, and classified under rigid requirements as to grading and moisture-content. Three wet classifiers were used to control the sand-grading and to bring the fineness-modulus to about 2·70. The material retained on a scalping screen with 8-inch square openings was crushed and recirculated, and that passing the 8-inch screen was separated by vibrating screens of $2\frac{3}{4}$-, $1\frac{1}{2}$-, $\frac{3}{4}$-, and $\frac{1}{4}$-inch square openings. The five products were transported by rail from stock piles to the storage bins at the concrete-mixing plants.

The two mixing plants, designated "Lo-mix" and "Hi-mix" plants on account of their locations at different levels in the canyon, are of especial interest because of their large capacity, remarkable performance, and the incorporation of novel features since duplicated in other large plants.

The operating controls of the two mixing plants were similar and to a large extent automatic. Cement, water, sand, and the four sizes of gravel were fed simultaneously to individual, full-reading, dial-equipped weighing batchers, which discharged in controlled sequence

Fig. 3.

BOULDER DAM.

ecial Lect.

Fig. 4.

GENERAL VIEW OF CANYON AT DIVERSION-TUNNEL OUTLET PORTAL.

Fig. 5.

TRUCK-MOUNTED DRILL-RIG USED IN UPPER PORTION OF DIVERSION TUNNELS.

to conveyors for ribbon-feeding to the mixers. The complete cycle of operations from the filling of batchers to the discharge of one or two mixers was controlled by a single operator. All mixers were of 4-cubic-yard capacity. Weighing was held within 1 per cent. of the desired amounts for cement and water, and within 2 per cent. for aggregates. A combined automatic recorder, visible to the operator, made a graphic time-record of the complete batching operations, in addition to indicating the consistency of the concrete.

The "Lo-mix" plant attained a production record of 2,462 cubic yards in 8 hours, with four mixers operating. The "Hi-mix" plant made a similar record of 3,001 cubic yards with six mixers operating.

Main Features of Project.

The main features of the Boulder Dam project include the diversion tunnels, spillways, dam, intake towers, penstocks and outlet conduits, and the power plant. Some of these features required the solution of unusual design problems because the structures were of such unprecedented dimensions or were subjected to such unusual conditions as to raise serious doubts about the adequacy of conventional design methods. A few of these special design problems will be cited in describing the various features of the project. A plan of the works is shown in Fig. 2, Plate 1, a photograph of the dam being shown in *Fig. 3* (facing p. 164).

Diversion Tunnels.

The construction of the project naturally had to start with the four diversion tunnels. Located in pairs, on both sides of the river, these tunnels are 50 feet in diameter and average about 4,000 feet long. They are lined with 3 feet of concrete and therefore the tunnel excavation has a diameter of 56 feet.

One of the more difficult questions to be decided in connexion with these diversion tunnels was the economic feasibility of excavating a 56-foot hole through the andesite breccia rock of this dam-site. It was an important decision because the four 56-foot tunnels total over 3 miles in length and the cost would have been prohibitive had bad tunnelling-conditions been encountered. The rock proved to be unusually sound and monolithic with the result that 1,500,000 cubic yards of excavation in the 3 miles of tunnel were removed without the use of timbering or roof supports of any sort. The ideal character of the andesite breccia rock for tunnelling purposes, as evidenced by this record, is one of the marvels of Boulder dam.

The initial tunnel operations included the driving of adits from the canyon walls to intersect the tunnels about mid-way of their lengths.

From these adits, 12-foot by 12-foot pioneer top-headings were driven both ways toward the portals.. At the same time top-headings, 41 feet high and the full 56-foot width of the tunnel, were started from the portals. After meeting the pioneer top-headings each tunnel excavation was enlarged to a 41-foot by 56-foot section. The bottom 15-foot bench was excavated just ahead of the final trimming and scaling operations. Fifteen hundred men working from eight tunnel faces excavated a maximum of about 455,000 cubic yards in one month. The outlet portals are shown in *Fig. 4* (facing p. 165).

In excavating the 41-foot by 56-foot section, a truck-mounted drill-carriage (*Fig. 5*, facing p. 165) was used at each tunnel face with from 24 to 30 drills working simultaneously. One-half of the tunnel face was drilled with one setting of the carriage. After the drilling operation was completed the drill-carriage was moved out of the danger zone, the holes were loaded, and the whole round of blasts was set off at one time. The tunnel muck was removed by electric power-shovels and trucks. Records kept for 1 week showed an average drilling time of 4·4 hours, an average mucking time of 6·5 hours, and a total average time per round of 14·9 hours. The average advance for each round was about 15 feet and the volume amounted to about 860 cubic yards. This rate was increased later so that at some faces three rounds were completed in 24 hours.

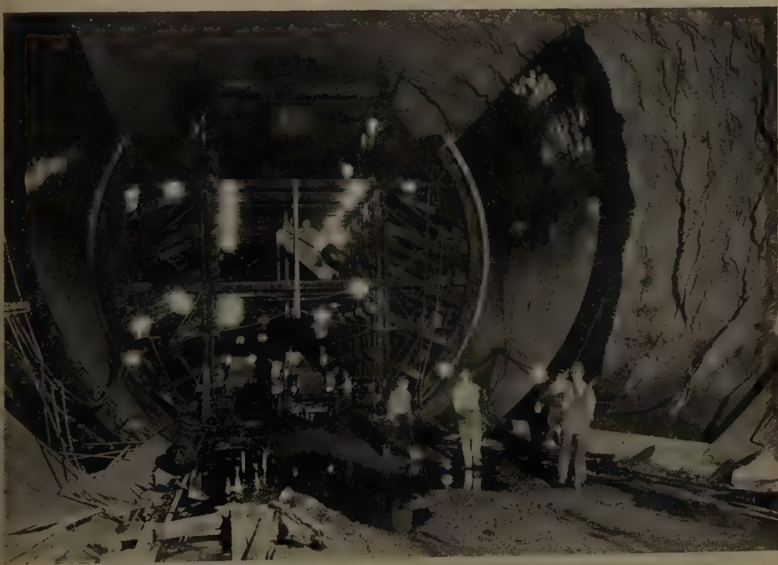
The first operation in preparing to place the invert concrete was the construction of two longitudinal concrete strips to serve as rail bases for the gantry crane and as supports for the steel side-forms. These strips were below the minimum thickness line and were later covered by the side-wall concrete. The gantry crane travelled back and forth picking up buckets and dumping them where desired. The crane had a maximum travelling speed of 300 feet per minute and a hoisting speed of 100 feet per minute. The invert surface was made to conform to the desired curvature by a template mechanically operated and running along the upper flanges of the side forms. Concrete-placing was started in the bottom, and as the invert filled up the concrete was moved out towards the edges by travelling screeds. Additional hand finishing was done by workmen stationed on a travelling platform supported on the side forms. The operation of placing is shown in *Fig. 6*.

The side walls were placed some little time after the invert. Two concrete strips were placed on the invert to serve as rail supports for the side-wall jumbo. The jumbo, a mammoth structural-steel framework 80 feet long, 50 feet high, and weighing 385 tons, was moved into position and then adjusted by means of screw-jacks and ratchets. Six rows of shoots were provided on each side, leading into openings in the forms. The dump buckets were picked up by the bridge crane

Fig. 6.

PLACING INVERT CONCRETE IN DIVERSION TUNNEL.

Fig. 7.



PLACING SIDE-WALL CONCRETE IN DIVERSION TUNNEL.

Fig. 8.

CONCRETE-GUN CARRIAGE FOR TOP-ARCH CONCRETE IN DIVERSION TUNNELS.

Fig. 9.

ARCH-FORM CARRIAGE FOR TOP-ARCH CONCRETE IN DIVERSION TUNNELS.

at the top of the jumbo and emptied into the shoots, from which the concrete slid into position back of the form without dropping any appreciable distance. Before the concrete reached the bottom of a form-opening, a cover was bolted into place. Concrete puddlers and an inspector were stationed behind the forms. About 50 hours were required to complete two 40-foot sections of the side wall and set up for another section. This included 24 hours for placing concrete, 10 hours waiting to remove forms, and 16 hours for moving and resetting the jumbo. *Fig.* 7 (facing p. 166) shows the method of placing the side-wall concrete.

The top or arch section was placed pneumatically. The structural-steel jumbo that supported the forms and carried all of the essential apparatus was a three-part structure, mounted on wheels and travelling on the same rails used by the side-wall jumbo. There was an arch-form support, a concrete-gun carriage, and a pipe carriage. The forms were moved into position and adjusted by screw-jacks. The pipe carriage, stationed between the arch-form jumbo and the gun carriage, supported the 6-inch pipe through which the concrete was conveyed to the forms. Two 2-cubic-yard concrete guns with hoppers were mounted on the gun carriage. Concrete was transported in 4½-cubic-yard agitators by trucks. The agitators were hoisted and discharged into the hoppers and the concrete was shot upward and forward through the delivery pipe. Placing was started at the end farthest from the gun carriage, the equipment being moved backward as the work progressed. The gun and arch-form carriages are shown in *Figs. 8* and *9*.

The concrete lining was placed in three different operations; the invert arc of 74 degrees, the side-wall arcs of 88 degrees each, and the top arc of 110 degrees. The standard length of each section placed was 40 feet. Concreting was carried forward from the upstream portals with the concrete transported by trucks from the "Lo-mix" plant in agitators or dump buckets. A maximum of 66,000 cubic yards per month was placed in the tunnel-lining. The concrete was cured by spraying the surface with a coat of asphaltic compound to prevent evaporation losses.

The tunnels were grouted under pressures of 100 lbs. per square inch to fill voids above the arch-lining, and certain portions were grouted under pressures up to 500 lbs. per square inch to seal and consolidate the rock surrounding the tunnel-lining. Drainage holes were drilled through the lining in portions of the tunnels not under internal pressure.

After serving their purpose in diverting the river, the tunnels were plugged with concrete about midway of their lengths. The portions downstream from the tunnel plugs in the two outside tunnels were

used for spillways, and in the inside tunnels these portions were utilized for the installation of steel penstocks leading to the power turbines and outlet valves. The plug outlet works are shown in Figs. 10, Plate 1.

Cofferdams.—The cofferdams were earth- and rock-fill dams of ordinary design and construction. The upstream cofferdam was protected on its upstream face with reinforced-concrete paving terminating in a steel-pile cut-off at the upstream toe. The downstream cofferdam had no such provisions but was protected from the back-wash of tunnel flow by means of a heavy rock barrier located between the cofferdam and the tunnel outlets. The upstream cofferdam was about 90 feet high and the downstream one was about 60 feet high. There was no difficulty in keeping the 125-foot deep excavation dry as the maximum seepage did not at any time exceed 2,000 gallons per minute. The percolation slope from the high-water surface above the sheet-pile cut-off at the upstream cofferdam to the bottom of the deep excavation was 6 to 1. This percolation slope was probably flatter than necessary for the stable gravelly material encountered, and it is interesting to note that after the experience at Boulder dam the Bureau engineers adopted a percolation slope of 3·65 to 1 for the 250-foot excavation at the Parker dam located on the Colorado river about 150 miles downstream below Boulder dam.

Spillways.

The available flood data for the Colorado river indicated that the maximum flood in the period of record since 1860 reached about 300,000 second-feet. The data, however, pointed to the possibility that even larger floods had occurred during this period.

Based on these data and other considerations, it was decided to provide a total capacity of 520,000 second-feet through the spillways, outlet works, and power plant. This total capacity was divided; 400,000 second-feet through the spillways, 100,000 second-feet through the outlet works, and 20,000 second-feet through the power plant.

The Boulder reservoir, now designated "Mead Lake" in honour of the late Dr. Elwood Mead, has a capacity of 30,500,000 acre-feet, of which 9,500,000 acre-feet have been reserved for flood-control. The ponding effect of this flood storage, combined with the 520,000 second-feet of flood-discharge capacity, provides for an estimated inflow into the reservoir of nearly 1,000,000 second-feet without overtopping the dam. This extraordinary provision for inflow into the reservoir is predicated on the remote possibility of an upstream-dam failure.

The design of the spillways was governed to a large extent by the fact that the two outside diversion tunnels were suitably located and

of proper size to be utilized as outlet conduits for the spillway discharge. The two overflow channel-spillways, one on each side of the canyon, were therefore located directly over these tunnels and connected to them by inclined shafts. The two spillways are identical in capacity and in all structural details except as effected by the topography.

Each spillway has a net weir-length of 400 feet and an over-all length of 700 feet from the upstream end to the portal of the inclined shaft. The weirs are constructed as massive gravity dams about 85 feet high and are surmounted by gate piers that rise 36 feet above the crest elevation. These piers divide the crest structure into four bays in each of which a 100-foot long by 16-foot high structural-steel drum-gate is installed. The drum-gate is a closed watertight segmental drum which is hinged at its centre of rotation to the concrete crest structure in such manner that it can be floated into the raised position or depressed into the lowered position by controlling the water-pressure in a hydraulic chamber in which the gate operates. The arrangement of the Arizona spillway is shown in Figs. 11, Plate 1.

The vertical drop of the water flowing down the inclined spillway-shaft is 550 feet and this results in a velocity of about 175 feet per second. Special research, including erosion tests on concrete surfaces under full-velocity conditions, were conducted to determine the effect of such a velocity on the concrete tunnel-lining. The available data and tests indicated that clear water flowing at this velocity would have no serious effect on sound concrete having a smooth and properly streamlined surface.

Probably the most important and unusual problem connected with the spillway design was the determination of the capacity of the combined channel and tunnel system. The spillway was first designed by analytical methods using formulas ordinarily applied to such problems and which had proved satisfactory for similar structures of ordinary dimensions. This analytical design was then subjected to careful model-tests to check its flow characteristics and capacity. The results of these model-tests showed unexpected and serious defects in this design and lead to the conclusion that a satisfactory design for a structure of such unprecedented dimensions could only be had through model-testing.

After eliminating several different structure types by model-testing and after testing several different modifications of the adopted type, models of the final design were constructed and tested in three different scales, 1 to 100, 1 to 60, and 1 to 20. The results obtained on all of these models were practically identical.

Two important structural features were incorporated in the final

design of the spillway channel as a direct result of the model-tests. One of these was a nearly vertical step 36 feet high at the downstream end of the channel, which acted as a weir or control that prevented objectionable flow conditions in the inclined shaft and tunnel. The other feature was an off-set or shoulder about 15 feet wide in the side of the channel adjacent to the spillway-crest structure. This off-set was located in the same plane as the pier face at the lower end of the downstream gate. It served as a side contraction for channel flow and was effective in eliminating spiral flow in the shaft and tunnel. These features are shown in *Fig. 12*, whilst *Fig. 13* shows the intersection of the inclined spillway-tunnel with the diversion tunnel.

The Dam.

Boulder dam is located in a narrow gorge approximately 1,000 feet deep, 300 feet wide at the base, and 870 feet wide at the top elevation of the dam. The canyon walls and rock floor are composed of andesite flow-breccia and andesite tuff-breccia. Silt, sand, gravel, and boulders overlaid the foundation rock to a depth of 120 feet. Although some minor faults are present at the site, there are no indications of recent movement and none of the faults are located where they affect the safety of the dam.

Although the rock is very satisfactory, exceptional precautions were taken in the treatment of the foundation and abutments. A comprehensive plan for low-pressure and high-pressure grouting was adopted. The low-pressure grouting was done prior to placing concrete, under pressures varying from 50 to 300 lbs. per square inch. The high-pressure grouting was done after placing at least 100 feet of concrete, using pressures from 500 to 1,000 lbs. per square inch.

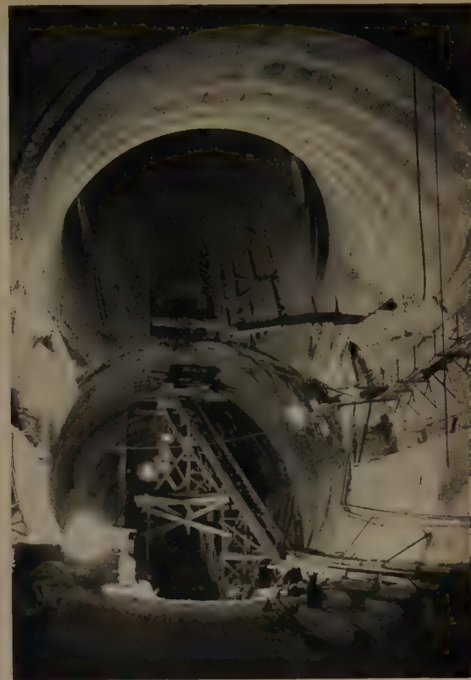
The dam was designed as a concrete arch-gravity dam. It is 45 feet thick at the top, 660 feet thick at the base, and 726 feet high from the lowest point in the foundation to the crown of the roadway (Figs. 14, Plate 2). This massive structure was divided into prismoidal columns or blocks by means of radial and circumferential contraction-joints to pre-determine the location of shrinkage cracks (*Fig. 15*, facing p. 171). The horizontal dimensions of the blocks vary from 25 feet to 30 feet at the downstream face to 50 feet by 60 feet at the upstream face. Each block is interlocked with adjacent blocks by vertical keys along radial joints, and by horizontal keys along circumferential joints. The radial joints are continuous through the dam while the circumferential joints are staggered at each block. The concrete in the dam was placed in 5-foot lifts.

An artificial cooling system was provided to remove excess heat, including that evolved by the cement in the hardening process, and to reduce the cooling and shrinking period to a relatively short time.

Fig. 12.

ARIZONA SPILLWAY AND DIVERSION-TUNNEL ENTRANCE.

Fig. 13.

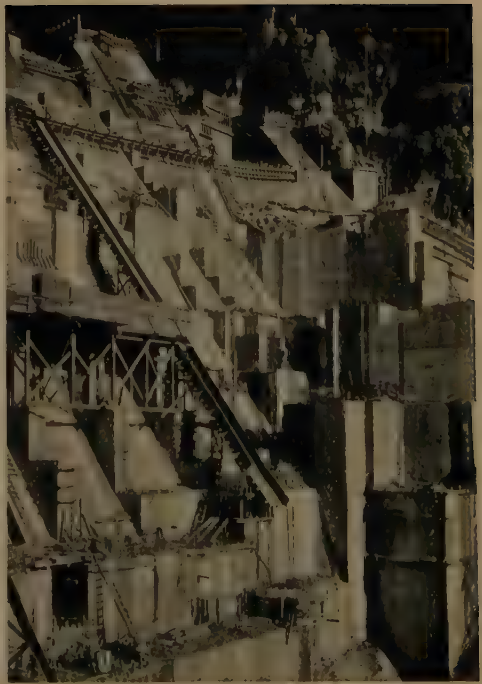


INTERSECTION OF INCLINED SPILLWAY-TUNNEL WITH DIVERSION TUNNEL.

Fig. 15.

CONCRETING OF LOWER PORTION OF DAM.

Fig. 16.



DOWNSTREAM FACE OF DAM, SHOWING FOOTINGS AND FOUNDATIONS FOR POWER PLANT.

The cooling was accomplished by circulating air-cooled and refrigerated water through 1-inch pipes embedded in the concrete. The dam was cooled a few degrees below the ultimate minimum temperatures expected, in order to allow for the small amount of chemical heat which might subsequently be generated and for the spreading of the canyon walls which might be caused by direct reservoir-pressures.

After each 50-foot height of dam had been cooled to the required temperature, the 8-foot cooling slot was filled with concrete to the top of the cooled section and the contraction-joints in the section were filled under pressure with cement grout. All cement used for joint-grouting was screened through a 200-mesh screen. A total of 33,500 bags of cement was used in grouting the contraction-joints of the dam.

Among the unusual problems requiring special investigation in the design of the dam were those pertaining to cement and mass-concrete. The most important of the mass-concrete problems was the control of volume change caused by temperature variations. Experimental studies resulted in the development of a satisfactory specification for the purchase of low-heat cement, which, in conjunction with the artificial cooling of the concrete and the comprehensive system of contraction joints, effectively solved the volume-change problem. These three expedients, together with a properly co-ordinated concrete placing-schedule, made possible the completion of the massive dam with practically no cracking. All contraction-joints have been effectively grouted and the structure is functioning monolithically as designed. Had ordinary cement been used and the dissipation of internal heat been left to natural processes, the dam would in all probability be shrinking and cracking for more than a century.

Bulk cement was purchased from five cement mills and the products of these mills were blended at the dam-site to insure uniformity in colour and general behaviour. The blending-plant equipment included eight steel silos with a combined capacity of 48,000 barrels of cement, interconnecting screw conveyors for blending the silo draw-offs, and pneumatic pumps for transporting the blended cement to the mixing plants.

The intelligent design and operation of the large-scale refrigeration plant required for artificially cooling the concrete necessitated the experimental determination of various thermal properties of mass-concrete. The results of the temperature studies, when checked against field observations, proved that laboratory values for thermal diffusivity, conductivity, and specific heat can be used accurately to predict temperature changes and movements in mass-concrete under field conditions.

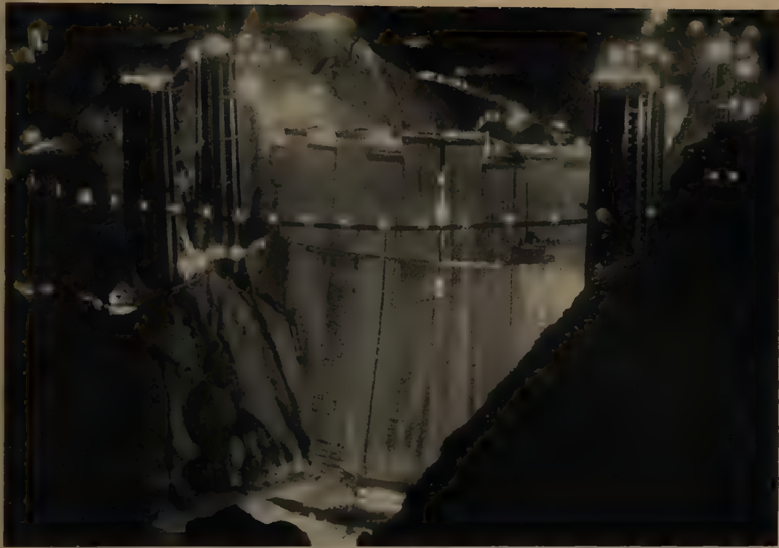
In the preliminary design studies for Boulder dam, it was attempted to proportion the dimensions so that the maximum stresses would not exceed 30 tons per square foot. Analyses of thirty-five tentative plans of widely-varying slopes, thicknesses, and radii of curvature showed, however, that owing to the great height of the structure such an ideal was impossible. Analyses of the adopted design, assuming a straight-line variation of stress, showed maximum cantilever compressive stresses of approximately 40 tons per square foot at the upstream edge of the base. However, the cantilever stresses at the downstream face of the dam, and maximum arch stresses in the horizontal elements were well below 30 tons per square foot. Resultants are at approximately the same location for both empty and full conditions of the reservoir, due to the great size of the structure and to the relatively flat upstream batter. The effect of non-linear distribution of stress was found to increase the arch and cantilever stresses along the planes of contact between the concrete and rock from 15 to 25 per cent. Earthquake stresses were considered, but necessitated no modifications in the adopted dimensions of the dam.

The analyses of stress conditions in the concrete and in the rock foundation and abutments were made by the trial-load method which brought the load deformations of the cantilever- and arch-elements into agreements at all points. The analyses included effects of direct water-pressures on the sides and bottom of the canyon as well as effects of loads transmitted by the dam. It was found that the spreading of the canyon walls due to direct water-pressures could be offset by from 1- to 3-degree increases in concrete-cooling. Special analyses and strain measurements made in a tunnel below the base of the dam showed that initial transverse compressive stresses in the rock strata were high enough to overcome the tendency for a longitudinal vertical crack to form along the bottom of the canyon.

The action of the dam and resulting stress conditions under temperature and water-loads were checked by detailed experimental investigations conducted on models as follows: two models of the entire structure; a slab model of the crown cantilever; and a slab model of an arch element selected at an elevation about half-way between the top and bottom of the structure. The models included appreciable depths of the foundation and abutment rock in all cases. One of the models of the entire dam and the two slab models were built of a plaster and celite mixture. The other model of the entire dam was built of a rubber-litharge compound.

The results of the model-investigations checked very closely with the results of the trial-load analyses. Both methods of study indicated the desirability of certain modifications in design details

Fig. 17.

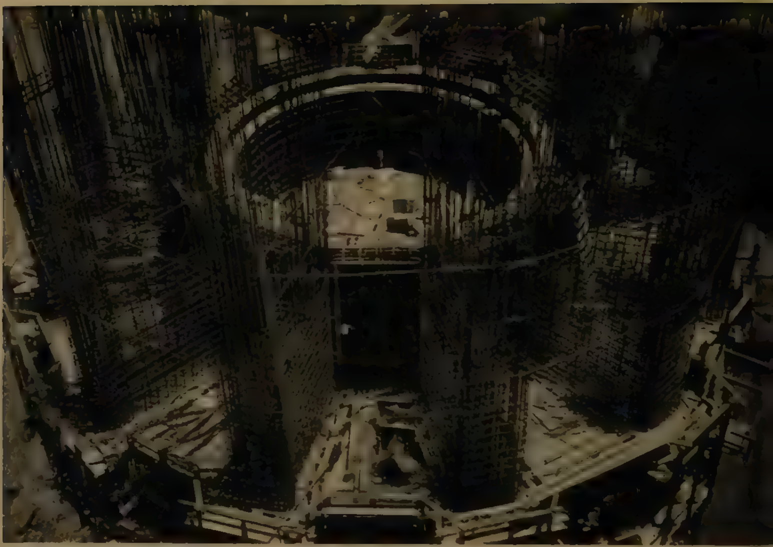


UPSTREAM FACE OF DAM AND INTAKE TOWERS DURING CONSTRUCTION.

Fig. 18.

CREST OF DAM FROM ARIZONA RIM OF BLACK CANYON.

Fig. 20.



TOP OF INTAKE TOWER, SHOWING INSTALLATION OF UPPER CYLINDER-GATE.

Fig. 22.

MANIFOLD AND CONDUITS LEADING TO TUNNEL PLUG OUTLETS.

which were later incorporated in the final plans. These included the addition of long-radius fillets at the downstream face of the dam near the arch abutments, for the purpose of reducing the compressive stresses at the intrados; and the addition of a short-radius fillet at the contact between the concrete and rock along the upstream face of the dam. Various stages in the dam are shown in *Figs. 15* to *18* (facing pp. 171 and 172).

Intake Towers.

The flow of water to the penstocks is controlled by four intake towers which are located in the reservoir upstream from the dam, two on each side of the canyon. Constructed of reinforced concrete, each tower consists of an inner barrel with twelve outside radial buttresses. The buttresses are required to stabilize the tower and to support the trash-racks. The outside diameter of the buttresses at the base is 82 feet and the total height of the tower, including the hoist house on top, is 394 feet. Girder bridges spanning from the crest of the dam and between towers give access to the four hoist houses.

Two cylinder-gates, each 10 feet high and 32 feet in diameter, control the flow of water into the barrel of the tower. The lower gate is located at the base and the upper gate at about midheight of the tower. Two sets of twelve emergency gates are provided to serve the four intake towers, one set being stored in one of the two towers on each side of the river. These emergency gates are designed to close the water ports in the tower and to permit inspection and maintenance of the cylinder-gates and other metal-work in the tower.

The structural design of the intake towers involved the consideration of earthquake stresses, which developed into the major design problem of this feature. Each tower required almost 4,000,000 pounds of reinforcing steel, which is evidence that the earthquake loads are enormous and could not be neglected with safety. From the standpoint of earthquake conditions, these towers are probably the most vulnerable structures of the Boulder Dam project, due to their great height and to their location in the water.

The hydraulic properties of the gates and towers were determined by laboratory tests on a 1-to-64 scale model. The discharge coefficient was found to be 0·90 for the lower gate and slightly higher for the upper gate. At full reservoir head and with both cylinder-gates open, about 60 per cent. of the total discharge of the tower passes through the lower gate. Three separate models were constructed of different portions of one of the four control systems to determine the hydraulic action in the combined tower and penstock and to measure losses. By the aid of piezometers installed throughout the models, it was possible to analyse the individual losses in the intake tower,

and to determine the losses in the main penstock and in the junction between it and the branches leading to the power plant. The arrangement of the towers is shown in *Figs. 19* and *20* (facing p. 173).

Penstocks and Outlet Conduits.

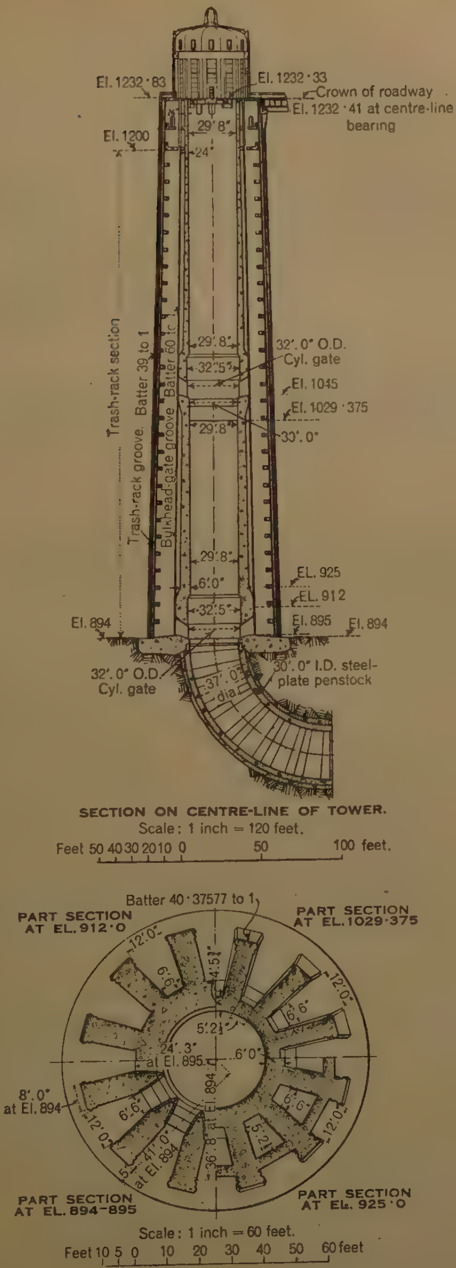
Steel penstocks and outlet conduits were provided to confine and distribute the water to the power units and irrigation outlets. The total capacity of the four penstock systems is 120,000 second-feet at maximum reservoir level, with all turbines operating and all outlet valves open. The discharge of 30,000 second-feet in each penstock system required a 30-foot-diameter header to a point immediately downstream from the four 13-foot power-penstock branches and a 25-foot header beyond this point to the needle-valve mainfold-piping.

Each 30-foot steel penstock-header connects with the steel throat-liner casting in the base of its intake tower and extends downward through a concrete plug in the inclined portion of the penstock tunnel. The penstocks that connect to the two upstream intake towers lead to the two inner diversion tunnels, the downstream portions of which are utilized as penstock tunnels. The penstocks that connect to the two downstream intake towers lead to two 37-foot-diameter penstock tunnels located at a level about 120 feet above the diversion tunnels. This arrangement provides two penstocks on each side of the river to supply power units and outlet valves. Details of the type of anchor used in the diversion tunnels below the power-penstock branches are shown in Figs. 21, Plate 2, whilst the manifold and conduits leading to the tunnel plug outlets are shown in *Fig. 22* (facing p. 173).

The conduits were designed for the maximum static head plus a computed water-hammer of 37 feet, based on a turbine-gate closure of 4 seconds and a relief-valve capacity of 80 per cent. Under this criterion the lower penstocks were designed for a head of 621 feet.

A design stress of 18,000 lbs. per square inch was used for the penstock plates, which were rolled from special carbon steel having a yield-strength of 38,000 lbs. per square inch. The maximum plate thicknesses for the different-sized pipes were $2\frac{3}{4}$ inches for the 30-foot, $2\frac{5}{16}$ inches for the 25-foot, and $1\frac{5}{16}$ inch for the 13-foot pipe. The specifications required electrically-welded, stress-relieved shop joints with an efficiency of 100 per cent. No field welding was permitted and all field girth-joints consist of an outside butt-strap with one end shop-welded and the other end field-pinned to the abutting pipe sections. Ground steel pins varying from $1\frac{1}{2}$ inch to $3\frac{3}{16}$ inches in diameter were used for plates over $\frac{7}{8}$ inch thick and hot-driven rivets were used for all thinner plates.

Figs. 19.



Vertical and Horizontal Sections of Intake Tower.

Fabrication.—Because of their enormous bulk, shipment of fabricated pipe sections was impossible. Consequently, it was necessary to erect a fabricating plant near the dam. The plant was fully equipped to fabricate, stress-relieve, and X-ray all sections. A total of 44,500 tons of steel plates was used in the 41,000 tons of finished pipe. The pipes were welded in lengths up to about 25 feet for the headers and up to about 50 feet for the 13-foot penstocks. Three plates were used for the 30-foot pipe, two for the 25-foot pipe, and one for all smaller pipe. The plates were rolled to the proper radius, tack-welded together, then welded into cylinders. The fillet-insert stiffener was next welded in between two cylinders and then the pipe section was completed by welding buttstrap and support brackets in place.

Automatic welding machines were used wherever possible. Each layer of deposited weld-metal was thoroughly cleaned and peened. Practically all welding was done with ¼-inch electrodes covered with a mineral-base flux. Specimens were tested for density of weld-metal, bending transverse to weld, yield, and ultimate strength. All main welds were X-rayed, using 300,000-volt portable X-ray units. Defective welds were chipped out and rewelded.

Completed pipes were stress-relieved by slowly heating in a furnace to between 1,100° and 1,200° F., soaking at this temperature, and then slowly cooling to 500° F. before removal. After stress-relieving, the ends of the sections were faced on the boring mill and pinholes subdrilled. All sections were shot-blasted on the inside and wire-brushed on the outside, prior to the shop application of priming paint.

Installation.—Completed sections, weighing up to 204 tons, were transported on a special trailer to the 150-ton cableway, and then lowered to landing platforms at the tunnel adit.[1] Specially-designed transfer cars carried the pipes through the adit and tunnel. The pipe sections were drawn together by a pulling rig consisting of three expanding spiders. Then the pinholes were drilled and reamed to size, leaving the holes about 0·001 inch per inch of diameter under-size to obtain a force fit. The plate was then trepanned by cutting an annular groove around the head of the pin and forcing the circular lip of metal under the head with a special caulking tool.

After erection, the pipes were pre-stressed with screw-jacks at girth joints left open between anchors. A compressive force equivalent to a temperature drop of 45° F. was applied. For this purpose, one hundred and forty-one mechanical jacks of 50 tons capacity each were used. The enlarged gaps in the pre-stressed joints were closed by tight-fitting segmental steel fillers.

[1] This was possible as the cableway was designed with an ample safety-factor to allow for occasional overloads.

Model-tests and photo-elastic analyses were employed where satisfactory analytical solutions were not possible. A 15-foot diameter pipe with a $2\frac{3}{8}$-inch shell and spherical heads was constructed for testing full-sized pinned girth-joints. Various models on a scale of 1 to 6 were tested, including typical pipe sections, the two-way Y, the three-way Y, and the 30-foot by 13-foot branch connexion.

Each of the completed penstock systems was subjected to hydrostatic pressure tests at about 150 per cent. of maximum operating head, plus allowance for water-hammer. For this purpose, a 30-foot diameter demountable test-head was pinned into the upstream end of the level portion of the 30-foot header. A few minor cracks which developed during these tests were repaired by welding and the pipe was retested. The pipes were painted with coal-tar primer and enamel on the inside, and aluminium paint on the outside. The steel penstocks were furnished by the Babcock & Wilcox Company of New York City at a total cost to the Government of $12,600,000.

Power Plant.

The power-plant building is of reinforced concrete with structural-steel roof girders, interior columns, and interior beams. Crane girders are reinforced concrete and are designed for two 325-ton cranes. The roof was designed to withstand safely a 2,000-lb. rock falling 300 feet. Generator-room floors were designed for a live load of 1,000 lbs. per square foot. Floors in the main parts of the building are terrazzo finish with coloured Indian designs. All railings, windows, exterior doors, and principal interior doors are aluminium, chemically treated, so as to present a pleasing contrast with the concrete walls. A general plan of the plant is shown in Fig. 23, Plate 2.

While this great power plant is all under one roof, it really comprises several large generating stations, each operating independently and supplying power to a separate system. One group of four of the large units will supply power to the Metropolitan Water District for operating the pumping plants on the Colorado river—Southern California aqueduct; a second group will generate power for the cities of Los Angeles, Burbank, Glendale, and Pasadena; a third group will supply power to the Southern California Edison Company; and the two 40,000-kilovolt-ampere generators will supply power to the Southern Sierras Power Company and the Los Angeles Gas and Electric Corporation. Eventually there will be still another group to furnish power for use in the States of Arizona and Nevada.

The power plant is designed for an ultimate installation of fifteen 82,500-kilovolt-ampere main generators, two 40,000-kilovolt-

ampere main generators, and two 3,000-kilovolt-ampere station service generators. This makes a total ultimate installation of 1,323,500 kilovolt-amperes.

Turbines.—The turbines are of the vertical-shaft, single-runner, reaction type with cast-steel spiral casings and runners. Each of the large turbines is designed to produce 115,000 HP. when operating under an effective head of 475 feet. The operating head will vary from 420 to 590 feet. The turbines operate at a speed of 180 revolutions per minute. The casing has twenty-four fixed stay-vanes cast integral with the speed-ring and twenty-four movable cast-steel wicket-gates. Each casing was subjected to a hydrostatic test at the factory of 500 lbs. per square inch, which is nearly twice the maximum operating pressure. A pressure of 225 lbs. per square inch was maintained in the casing while the surrounding concrete was placed, which allowed the casing to expand and assume the shape it will have under normal operating conditions. This avoided stressing and cracking of the concrete due to expansion of the casing under pressure.

In order to secure accurate running-clearances between the runner and the casing, the seats for the stationary wearing rings were machined by means of a portable motor-driven boring tool after the casing was erected in the power plant. Each turbine is provided with a pressure-regulator or relief valve having a discharge capacity of 80 per cent. of the full-gate turbine-discharge. The pressure-regulator valve is mechanically connected to the turbine-gate operating mechanism and may be operated either as a synchronous by-pass or as a water-saving valve. Normally the pressure-regulator is operated to save water and is arranged to open as the turbine gates close upon loss of load, and the pressure-regulator then slowly closes under control of an oil-filled dash-pot. The energy in the jet from the pressure-regulator is dissipated in an energy-absorber located immediately below the pressure-regulator.

The speed of each turbine is controlled by a governor of the oil-pressure relay-valve type, with the speed-responsive fly-balls driven by a small synchronous motor receiving its power supply from a permanent-magnet alternating-current generator connected to the main shaft of the generator. The speed-responsive element is sensitive to speed changes of $\frac{1}{100}$ of 1 per cent. The governors operate with an oil pressure of 300 lbs. per square inch.

Generators.—The generators are of the vertical-shaft, water-wheel-driven type with main exciter, pilot exciter, and permanent-magnet generator for operating the speed-responsive element of the turbine governor, all direct-connected to the upper end of the main shaft. Each of the large generators has a rated capacity of 82,500 kilovolt-

amperes at unity power-factor, and operates at 180 revolutions per minute, 60 cycles, and 16,500 volts. The generator has a fly-wheel effect of 110,000,000 lbs. at 1 foot radius, and low reactance and high short-circuit ratio to increase the stability of the transmission system. The generators are provided with an enclosed air-circulating system. Eight surface coolers, through which cold water is circulated, serve to cool the air before it re-enters the generator. The rotor is 25 feet in diameter, weighs 580 tons, and is handled by a lifting beam and four 150-ton hoists on two 325-ton overhead electric travelling cranes. A cross-section through a 115,000-HP. unit is shown in Fig. 24, Plate 2.

Each generator has two oil circuit-breakers which normally operate in parallel. Each breaker has, however, sufficient capacity to carry the full generator current. These breakers are of the high-speed, metal-enclosed, motor-operated type. They have a current capacity of 4,000 amperes, an interrupting capacity of 2,500,000 kilovolt-amperes, and operate in 8 cycles.

Low-voltage Bus-bars.—The bus-bars or conductors connecting the generators and transformers and the transfer bus-bar for substituting a spare generator, are of the metal-enclosed type. The conductors consist of two 6-inch copper channels mounted with the flanges inside to form a box section. The bus-bars are held by rugged porcelain insulators at 6-foot intervals and are completely enclosed by copper plates ½ inch in thickness.

Transformers.—Normally two generators and one bank of transformers are operated as a unit. The transformers are of the single-phase, oil-insulated, water-cooled, outdoor type and each transformer has a normal rated capacity of 55,000 kilovolt-amperes. The transformers raise the voltage from 16,320 volts to 287,500 volts. They are connected delta on the low-voltage side and Y or star on the high-voltage side. The transformers are provided with a cushion of nitrogen gas above the oil.

Cubicle.—Local control for each generator is afforded by means of a cubicle located in the generator room. This cubicle contains all the necessary control switches and instruments for starting, stopping, and controlling the unit. In addition to providing an emergency operating station, the cubicles contain all relays, meters, voltage regulator, and other equipment not essential in the remote-control room.

Control Boards.—The main control boards are located in the control room in the central section of the power-house. The control of the generating units can be transferred from the generator cubicles to the remote control board, or vice versa, by means of control transfer-switches. The main control boards are of the miniature type with

indicating instruments 4 inches wide. Starting of the generating units is completely automatic. The operator pushes a button which first starts the lubricating pumps and cooling-water supply. Then the turbine wicket-gates open to the speed-no-load position, the unit starts and comes up to normal speed and excitation under control of the governor and automatic voltage-regulator, the speed is matched with the system speed, and the generator is synchronized automatically. Automatic load- and frequency control is provided.

High-voltage Switching Station.—The high-voltage switching stations to which the outgoing transmission circuits connect are located near the Nevada canyon rim. The several banks of step-up transformers, which are located on the platform along the river side of the generating room, are connected with the switching station by overhead high-voltage circuits supported on steel structures. The switching station and the circuits connecting the transformers with the switching station are protected from lightning by an overhead diverter system. The almost vertical spans from the power-plant to the canyon rim are provided with weights and anchor cables passing over sheaves to maintain uniform tension under different temperatures. The towers at the canyon rim are inclined 30 degrees from the vertical to afford necessary clearance between the power-conductors and canyon walls. The switching-station apparatus and the power-plant transformers are protected by "Thyrite" station-type lightning arresters which function at 240,000 volts.

The high-voltage switching stations are of the double bus-bar type. Each outgoing transmission line and each power-plant circuit is provided with two 287,500-volt oil circuit-breakers of the conventional dead-tank, high-speed type, which have an operating speed of less than 3 cycles. Three breakers are solenoid-operated and have an interrupting capacity of 2,500,000 kilovolt-amperes. The oil circuit-breakers, and the disconnecting switches which are motor-operated, are controlled from the main control room in the power plant. The control wiring between the power plant and the switching station is installed in an inclined tunnel.

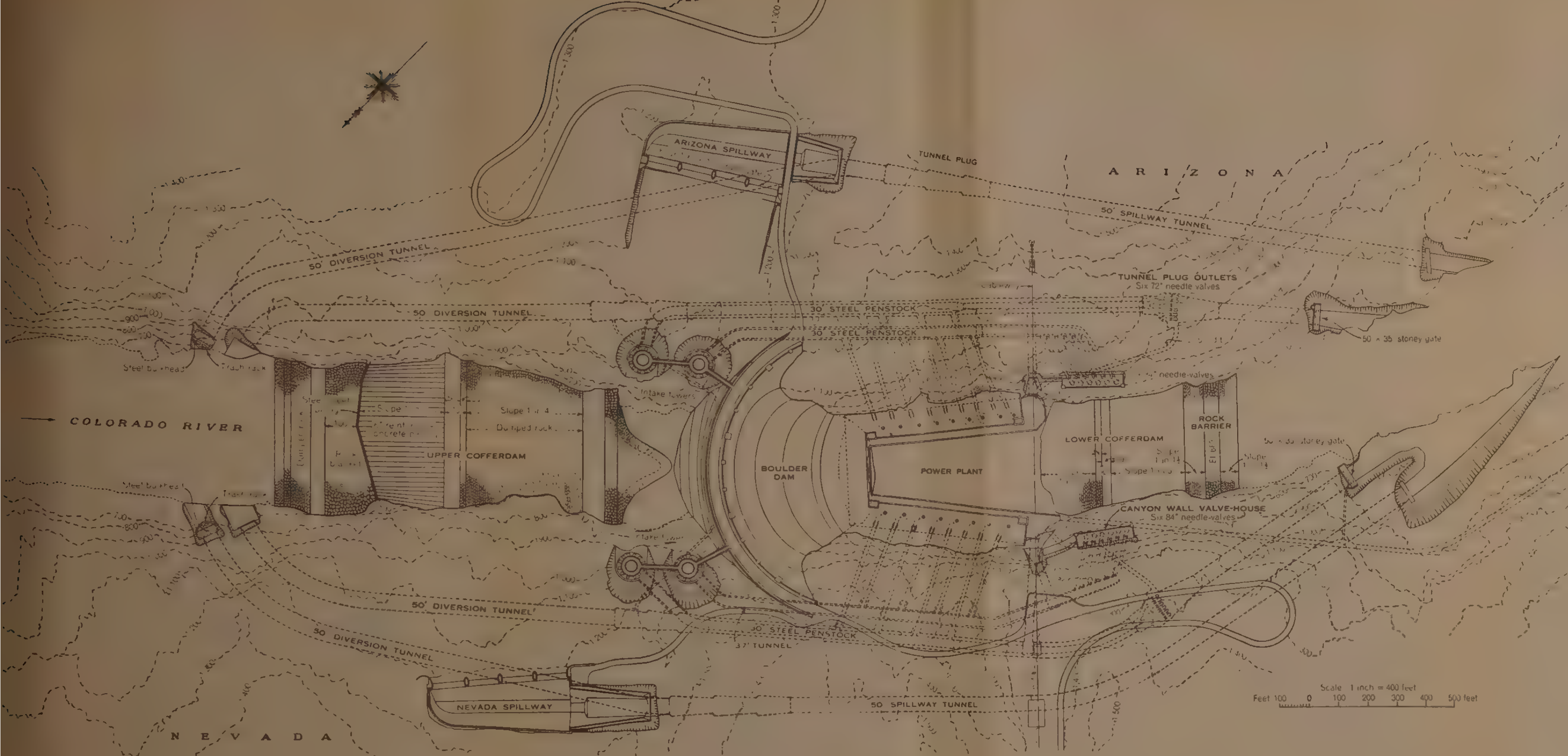
The Lecture was illustrated by lantern-slides and a cinematograph film.

The Meeting concluded with a vote of thanks proposed by Mr. W. J. E. Binnie, Vice-President, and seconded by Mr. W. T. Halcrow.

⁂ A number of drawings, photographs and reports relating to Boulder dam have been received and have been placed in the Institution Library.—SEC. INST. C.E.

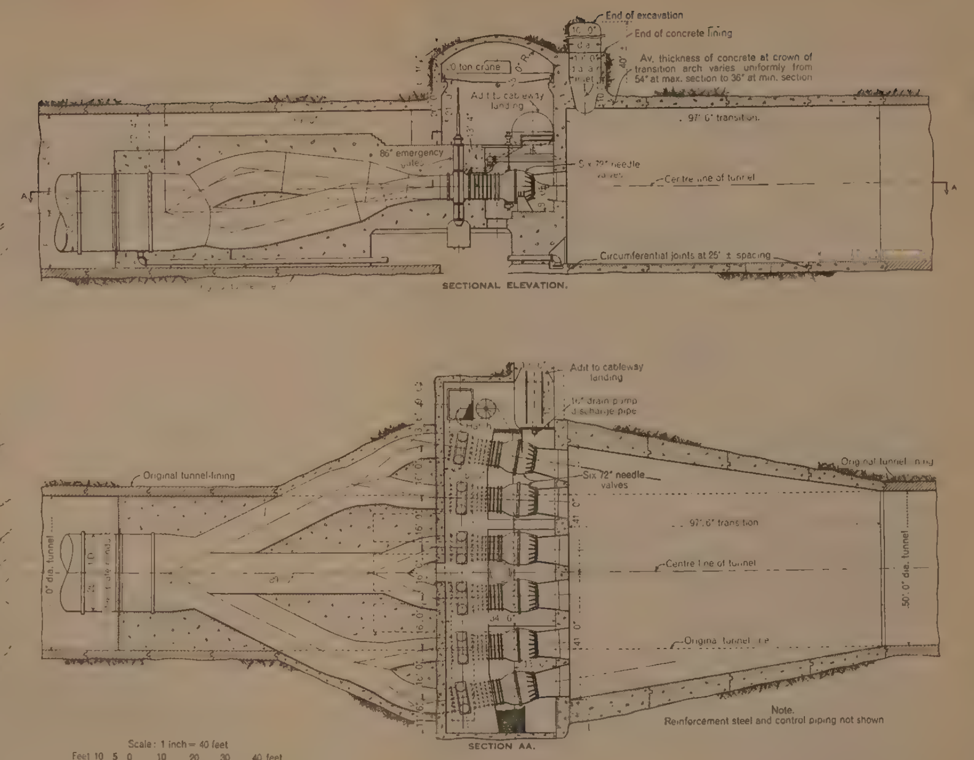
THE BOULDER DAM.

Fig: 2.



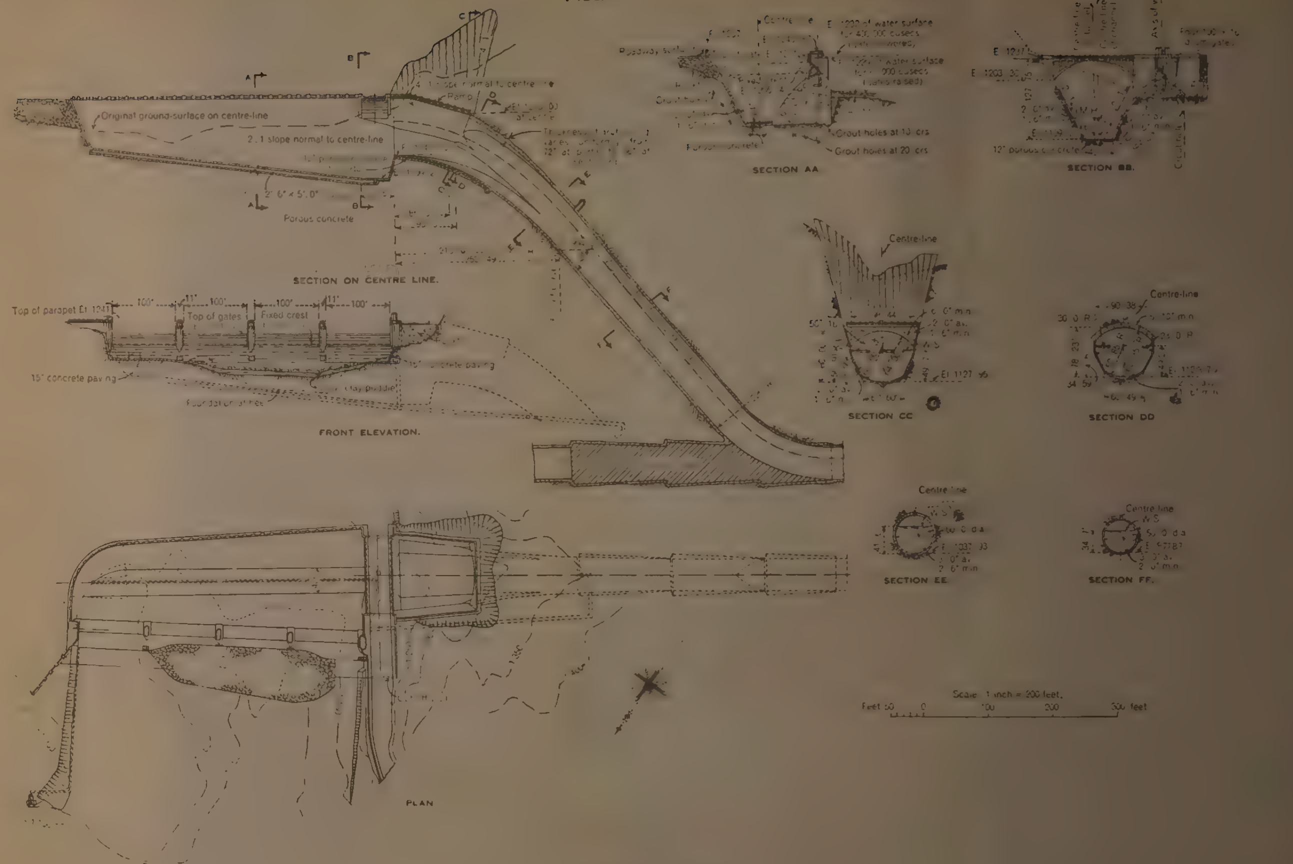
GENERAL PLAN OF WORKS

Figs: 10.



PLUG OUTLET WORKS.

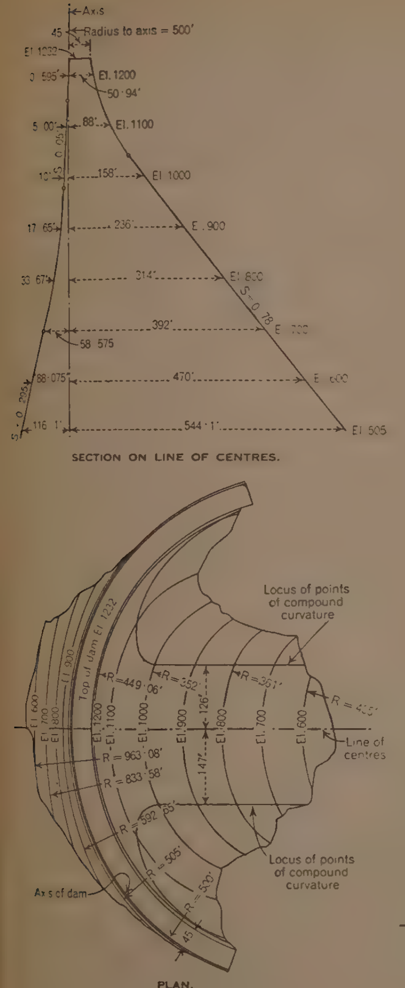
Figs: 11.



ARIZONA SPILLWAY.

THE BOULDER DAM.

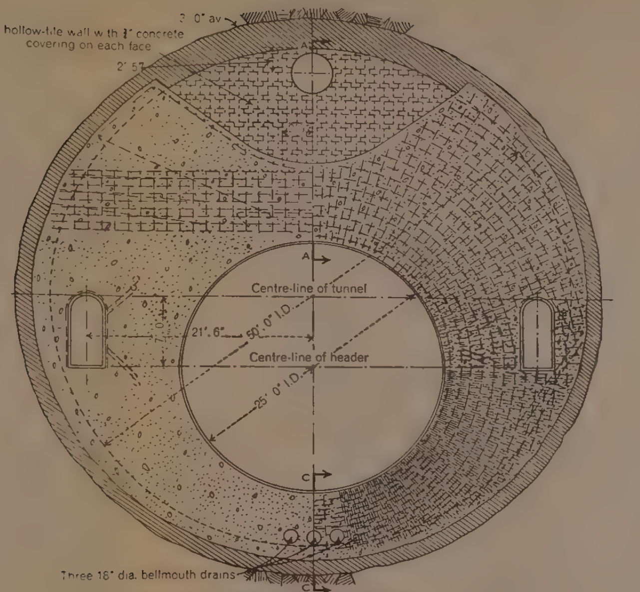
Figs: 14.



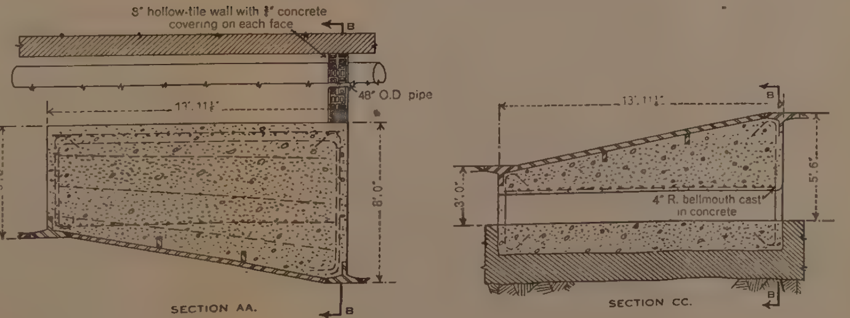
Scale: 1 inch = 320 feet.
Feet 100 50 0 100 200 300 feet.

SECTION AND PLAN OF DAM.

Figs: 21.



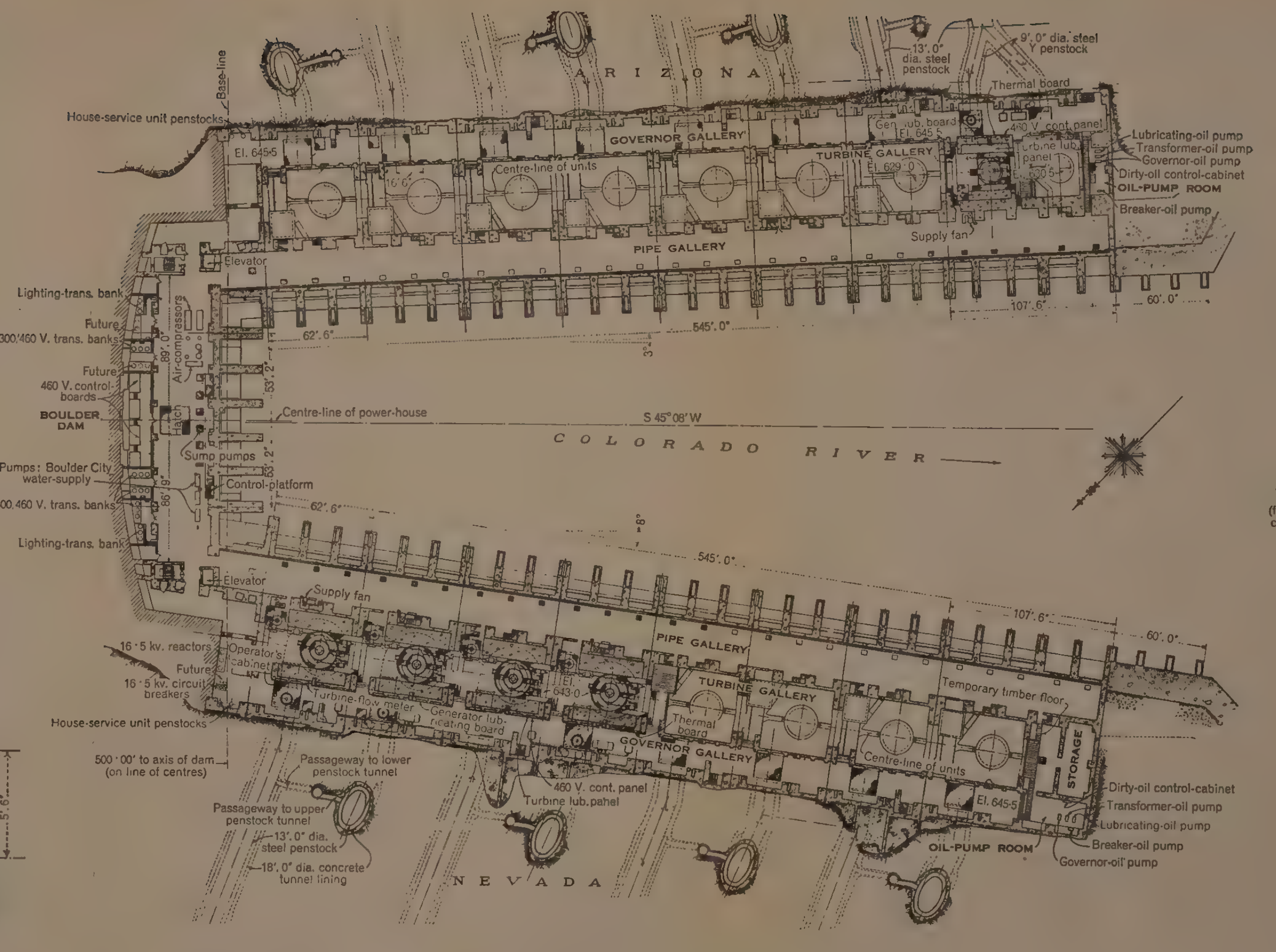
Arrangement of reinforcement and grouting units is symmetrical about vertical centre-line. Left half of this view shows arch passage-way and beam reinforcement. Right half shows circumferential and radial reinforcement.



Scale: 1 inch = 16 feet.
Feet 5 0 5 10 15 20 feet.

DETAILS OF ANCHOR IN DIVERSION TUNNEL.

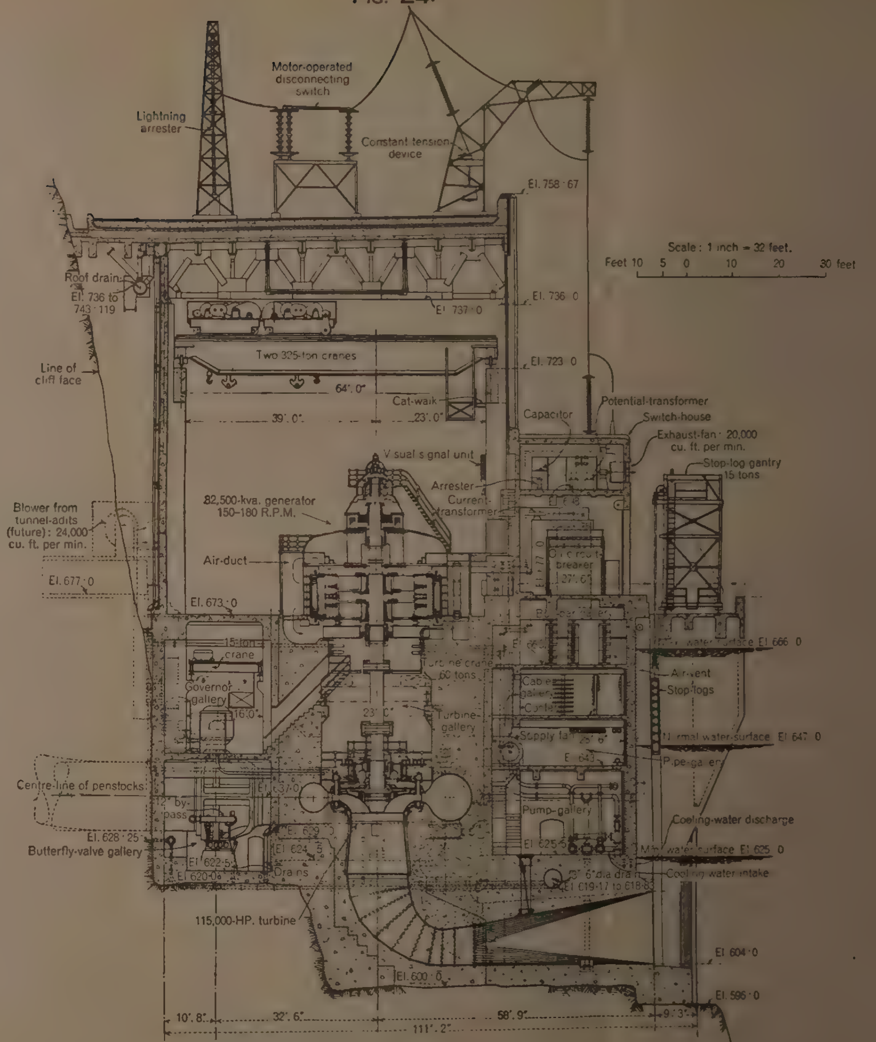
Fig: 23.



Scale: 1 inch = 100 feet.
Feet 10 0 10 20 30 40 50 100 feet

GENERAL PLAN OF POWER PLANT.

Fig: 24.



Scale: 1 inch = 32 feet.
Feet 10 5 0 10 20 30 feet

CROSS-SECTION THROUGH 115,000-HP. UNIT.

J. L. SAVAGE.

William Clowes & Sons, Limited: London.

EXTRA MEETING.

27 April, 1937.

SYDNEY BRYAN DONKIN, Vice-President, in the Chair.

THE JAMES FORREST LECTURE, 1937.

The CHAIRMAN referred to the fact that the James Forrest Lecture was established and endowed in the honour of James Forrest, who had been Secretary of The Institution from 1859 to 1896, and Honorary Secretary from 1896 until his death in 1917. The present Lecture was the forty-third of the series. Outside the lecture-theatre there were various pieces of presentation silver which had been received by Mr. Forrest from different societies and individuals. That silver was, by Mr. Forrest's special wish, exhibited on each occasion of the James Forrest Lecture.

He had pleasure in introducing to the members, although under the circumstances it was hardly necessary, Sir William Bragg, O.M., K.B.E., who would deliver a Lecture that evening on the subject of "The Crystal and the Engineer." Sir William Bragg was at present President of the Royal Society, and he had done The Institution the honour last November of allowing it to elect him an Honorary Member. The members of The Institution felt it a great honour that Sir William Bragg should deliver before them the James Forrest Lecture.

"The Crystal and the Engineer."

By SIR WILLIAM HENRY BRAGG, O.M., K.B.E., D.Sc., P.R.S., Hon. M. Inst. C.E.

THE structure of every crystal contains the solution of an engineering problem. The atoms of which the crystal is made are held together in some regular pattern. The forces which hold them form a set of stresses which maintain an interior balance, and at the same time resist up to yielding point any stresses that may be applied from without. The design of a crystal may be likened to that of a bridge, the component parts of which are held together by ties and struts of calculable strength. Just as the engineer uses his knowledge of the properties of steel and stone to find out the strength of a given design, so the student of the crystal may try to connect its observed characteristics with the disposition of the atoms and the forces that act between them. The engineer is probably the better equipped for his task, because in the first place it is not always an easy matter

to discover the crystal plan, and in the second place the laws of the atomic forces are not yet fully understood. The study of the crystal from what might be called the engineering point of view is, however, so interesting and so full of possible applications that even its difficulties and uncertainties are no deterrent. In fact, as the study proceeds it is to be expected that difficulties will in turn disappear and uncertainty will give place to better appreciation of natural laws. It would not be unreasonable to put the crystal problem before the engineer in the mere expectation that he would be interested in work so like his own in its nature, even though it is so widely different in scale.

To be sure, the structure of the engineer is put together for some definite purpose, but the structures of Nature are not always designed so particularly. A crystal of salt or the crystals in a piece of metallic ore or in a lump of clay do not, each one of them, perform a special duty for which form, size, and internal disposition have been calculated. Nevertheless, the arrangement of the atoms and the play of the forces, both within the substance and on the surface, give to it its characteristic qualities. For example, carbon atoms linked together in a certain way constitute the diamond; if the design is varied, graphite is the result. We naturally ask what the two designs may be, so that we may compare them and seek an explanation of the extraordinary difference between the two materials. Silicon and oxygen combine to make quartz, aluminium and oxygen to make corundum, ruby and sapphire, while silicon, aluminium and oxygen are the principal constituents of clays and felspars. What are the designs which are responsible for these very different compounds of simple elements?

On the other hand, Nature puts together the substances of the living organism with obvious intent that they shall each fulfil a definite function. For instance, the cellulose crystal, which is the chief constituent of all things that have roots in the earth, is long and flexible. At the same time it can withstand longitudinal stress. For these reasons it is obviously well suited to the needs of the living and growing plant, which indeed derives from it the characteristics of the plant-form. Muscle, nerve, hair, and generally the softer portions of the animal body are built of protein molecules which also are long flexible chains, and join in crystalline regularity to form substances that grow and bend and stand longitudinal stresses. Some of them can be extended and again be folded up, thus giving to muscle and hair their characteristic qualities. Others possess surfaces on which the arrangement of the atoms is favourable to catalytic or other physiological actions. In all cases arrangement according to some pattern is the fundamental condition.

Arrangement, when carried out on a scale sufficiently large, makes the visible crystal. In the vast majority of cases, however, arrangement does not go so far. It stops short while the aggregate is still beyond the vision of the eye, even when assisted by the microscope. X-rays, if we are to examine it, must be substituted for light; being some ten thousand times finer, they are able to show us the invisible details.

The subject of crystalline structure is therefore of interest to all who value the increase of natural knowledge, just because it is so fundamental, and the engineer may be especially interested because its problems are so like his own. But to the engineer the subject is more than academic, as indeed it is to everyone who works on the world's materials, living or dead. Since every substance used by the engineer is more or less crystalline and its properties depend on its crystalline state, he must be drawn to examine that state, to find out its details, and, so far as he can, to connect structure and properties. The problems to be solved may be very numerous and complicated, but it is necessary to attack them. How else can sound progress be made? The situation is very like that which is to be found in the world of medical science. The human body is extraordinarily complicated; one might well ask whether its details would ever be sufficiently well known and understood to give any command over health and disease. But if inquiry with microscope and other aids had been shirked there would have been no such improvements as we have seen even in our own time. Though the work to be done seems endless and the mass of detail terrific, yet reward comes by this and that discovery, not infrequently unexpected, and the future seems always more and more hopeful.

The same applies in the use of materials by the engineer; if X-rays and other forms of radiation show us details of natural structure which have been invisible hitherto, we see spread before us a wide field of knowledge which invites exploration. We have new means of study, by which we hope not only to learn but also to increase our command over materials, so that we may fit them to the work that we would ask of them, may know what to ask, how to ask, how to avoid asking too much or too little. Even a little of the knowledge may help us.

In all this new field the crystalline structure is the predominant feature. The X-rays have shown us how universal it is, how it is to be found not only in such obviously regular bodies as quartz or diamond, but even in wood and cotton, hair and wool, nerve and muscle. To quote from a recent lecture [1] by Dr. H. J. Gough and

[1] "Strength of Metals in the Light of Modern Physics." Journal Roy. Ae. Soc., vol. xl (1936), p. 586. (August, 1936.)

Mr. W. A. Wood " . . . the properties of any metal or alloy—whether we regard it merely as a solid body, a chemical combination of various elements or as an aggregate of crystals—*must* depend ultimately on its inner crystalline structure as revealed by *physical* methods." Of these methods, those that employ X-rays or electrons are the newest and the most powerful. What the engineer here asserts of metals and alloys is equally true of other substances—stone, pottery, cements, timber, ropes, textiles, paints, varnishes, and so on.

Let us consider first the metals and alloys, since these are of very great importance in constructional work. Assuming that we can examine by our new methods the lay-out of the atoms and molecules in a metal specimen, we search at once for explanations of the well-known metallic properties. What is the cause of the great variety in the properties of different metals, or of the same metal under different conditions ? Why do impurities so often alter the properties of a metal to a remarkable degree ? What is the nature of an alloy, and what again is the origin of the enormous variety of the different alloys ? How is it that it is found worth while to search blindly through the hundreds of thousands of possible variations in the hope of being rewarded by the discovery of some alloy of especially desirable qualities ? What is the real meaning of cold working ? How are the effects of temperature to be accounted for ? and so on.

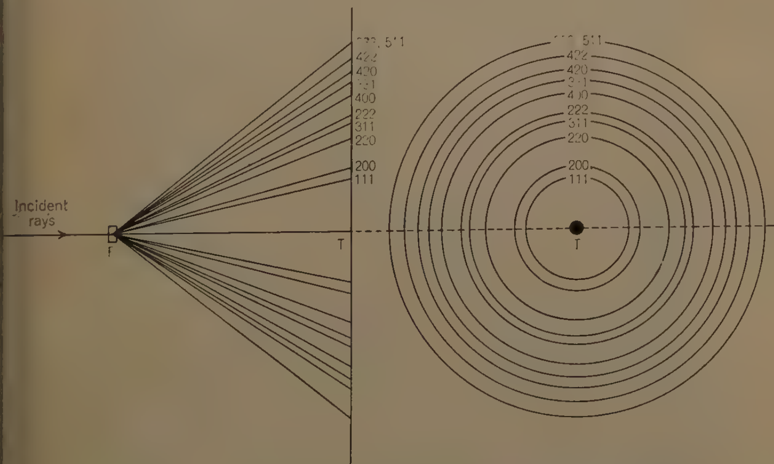
In the quarter of a century that has gone by since the first X-ray photographs of crystalline structure were made, an immense amount of work has been put into the search for answers to these questions. Answers have already been found for some of them, but not for all. In fact, the search has revealed as many questions that are new as answers to questions that are old.

An X-ray photograph may be of more than one kind, but the only kind which we need consider now is that which is obtained where a fine pencil of X-rays of single wave-length is made to strike the material under examination. The material is finely powdered, and its amount may be very small, no more than a few milligrams. In one form of the experiment the rays, after traversing the material, fall perpendicularly upon a photographic plate. On development the plate shows the spot where the rays struck it, and also, surrounding this spot as a centre, a number of rings which are due to the diffraction of the primary pencil by the crystalline material. Every different crystalline substance has its characteristic set of rings, by which it can at least be identified, and its structure can be calculated when the technical difficulties can be overcome. A set of rings due to aluminium is shown in *Figs. 1* and *2*. The photograph is of interest as it is an early example due to Hull, who was one of the first to employ the " powder method." A slight variation of the use of the flat

Fig. 1.

POWDER PHOTOGRAPH OF ALUMINIUM.
(HULL.)

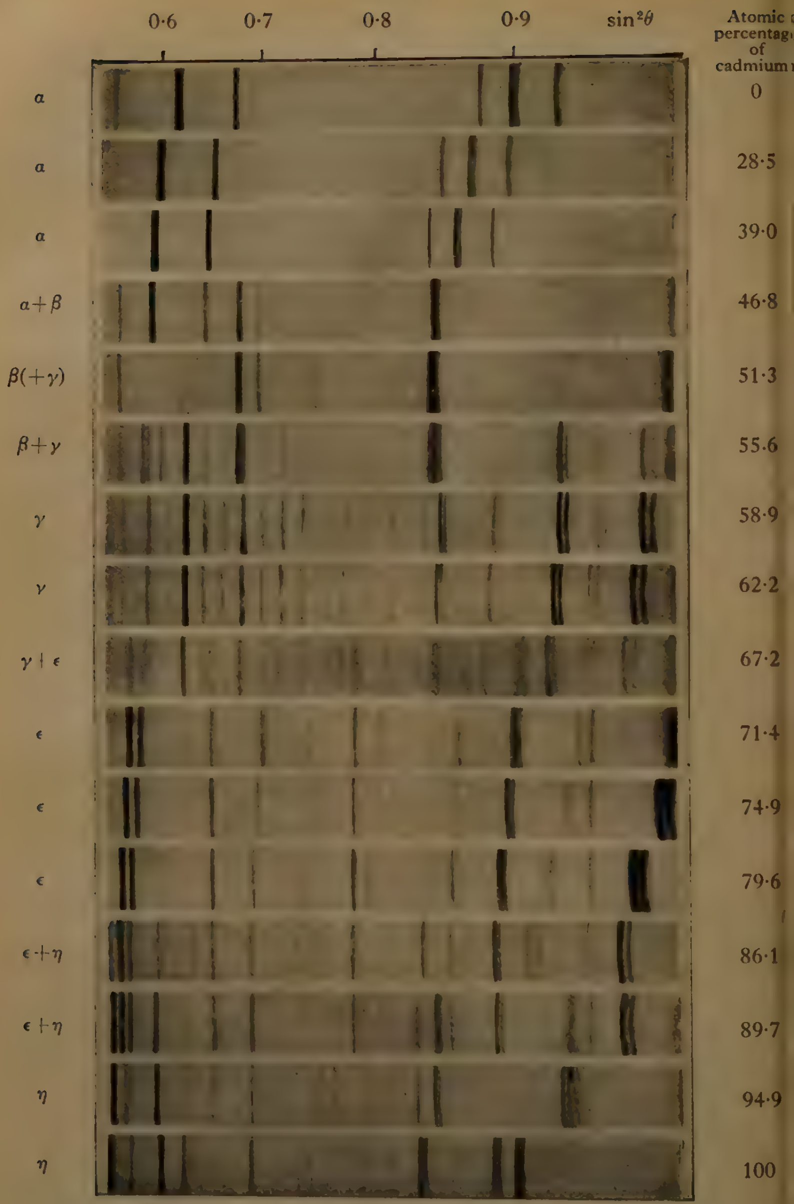
Fig. 2.



THE RINGS OF A POWDER PHOTOGRAPH ON A FLAT PLATE. THE POSITIONS OF THE RINGS ARE THOSE TYPICAL OF A FACE-CENTERED CUBIC LATTICE, OF WHICH THE ALUMINIUM STRUCTURE IS AN EXAMPLE.

F. Lect.

Fig. 4.



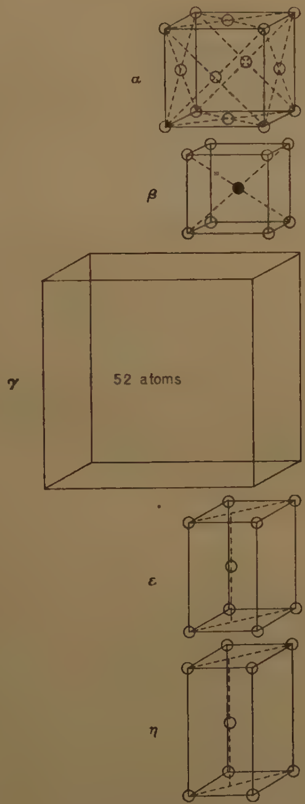
AN X-RAY EXAMINATION BY THE POWDER METHOD OF THE SILVER-CADMIUM ALLOYS. (WESTGREN.) FeK RADIATION.

(From a Paper by A. Westgren and G. Phragmén, "Gesetzmässigkeiten im Aufbau der Legierungen." *Metallwirtschaft*, vol. vii (1928), p. 700).

photographic plate is shown later. It is now usual to replace the plate by a strip of film disposed as shown in *Figs. 3* (p. 186).

If the powdered material has more than one component the corresponding spectra are superimposed without mutual injury.

Figs. 5.



STRUCTURE OF THE SILVER–CADMIUM ALLOY PHASES.

(From a Paper by A. Westgren and G. Phragmén, "Gesetzmässigkeiten im Aufbau der Legierungen." *Metallwirtschaft*, vol. vii (1928), p. 700).

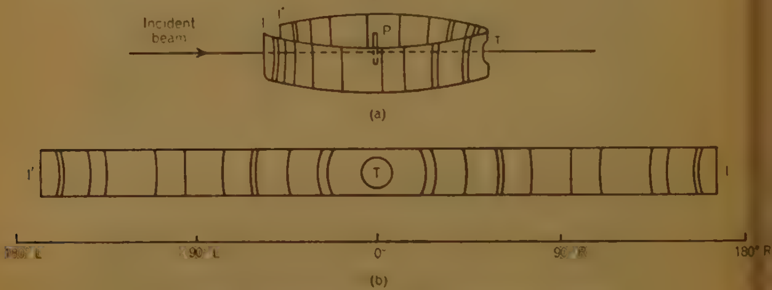
Consider, for example, the series of photographs shown in *Fig. 4*. At the top is the photograph due to pure silver. Certain lines are observed in it. From their positions and relative intensities the trained observer finds that the crystal of silver is of the form taken by a pile of spheres packed closely together, each sphere having a diameter of 2·9 Ångström units.[1] Several other metals such as copper,

[1] The Ångström unit is 10^{-8} centimetre.

gold, and aluminium possess the same structure and therefore yield the same photograph, except that the whole spectrum is displaced right or left according to the size of the atom. One close-packed metal is not confused with another, because the atoms vary in size.

At the bottom of *Fig. 4* is a spectrum of a different character; it shows that the cadmium structure is not that of a set of close-packed spheres, but is rather a set of close-packed spheroids. The argument is to be found in any book which goes fully into the subject; I do not give it here, because I am dealing only with results. The intermediate photographs relate to mixtures of silver and cadmium in various proportions, and it is to be observed that there are but a limited number of types of spectra, which overlie each other more or less. We have, in fact, set out before us the various alloys that are composed of silver and cadmium, or, to use the ordinary term, the

Figs. 3.



Powder Photograph on a Cylindrical Film.
(a) The film in position. (b) The film unrolled.

various phases of the silver-cadmium alloy. It is possible to determine with precision the proportions of the mixture when the metals are combined in any single alloy. If this is done at a variety of temperatures the metallurgist's phase-diagram is obtained. We have, therefore, a new way of determining phase-diagrams, which indeed possesses several advantages over the old method.

The separation of the spectra of two or more crystalline components is so clear that it is considered to be possible to proceed to the more difficult problem of the ternary alloy; no progress to its solution has been made by the older methods. Obviously this new power is of great interest and importance. The great variety of the properties of different alloys invites the search, as I have said already, for alloys that will fulfil the extraordinary demands of modern constructions, and any method of classification such as that now coming into view is sure to shorten the work. Some ternary alloys have been discovered that are of great usefulness, and there must be many such, ternary, quaternary, and so on, which are as yet unknown.

From the appearance of the spectrum of any substance it is possible, in theory at least, to discover the structure, that is to say, the arrangement of the atoms in the unit of pattern, although sometimes the complications are great and the argument difficult. All the phase-structures of silver-cadmium are known. The α-phase (see the Greek letters to the left of the photographs in Fig. 4) is that of close-packed silver; as cadmium atoms are added they take the place of some of the silver atoms without breaking up the arrangement. They do, however, swell the structure somewhat, as is shown by the gradual shift of the spectrum to the left. When the proportion of cadmium exceeds about 40 per cent. a new pattern appears—the β-phase—in which there are equal numbers of cadmium and silver atoms. Each cadmium (or silver) atom lies at the centre of a cube at the corners of which are eight silver (or cadmium) atoms; the two modes of description are equivalent. As the proportion of cadmium is again increased the β-phase dies out, and a new and most interesting phase makes its appearance. The so-called γ-structure is much more complicated than that of the α- or β-phase. It was first worked out by Messrs. A. J. Bradley and J. Thewlis [1] who showed that its unit cell contained the unusual number of 52 atoms, 20 being silver and 32 cadmium. A still greater excess of cadmium leads to the ϵ- and η-structures, of which the last is that of pure cadmium. The relative sizes of the units of pattern of the different phases are shown in *Figs. 5* (p. 185).

Thus the X-ray photographs give much information about the phase of an alloy. Mr. W. Hume-Rothery has observed that a most interesting conclusion can be drawn from a comparison of the properties of phases belonging to different series. For instance, there occurs in several other cases the same curious γ-phase, with its pattern composed of 52 atoms, and its hard, brittle, intractable nature. It is found in brass, in an alloy of copper and aluminium, in one of copper and tin, and so on. At the first glance there seems to be no rule governing the proportions of the constituents in the different cases. But Mr. Hume-Rothery pointed out that a very simple rule connects the total number of atoms in the cell with the number of valency electrons. Thus the unit cell of γ-cadmium-silver contains the chemical molecule Ag_5Cd_8 repeated four times. Now, each atom of silver contributes one valency electron, and each atom of cadmium two, so that the molecule Ag_5Cd_8 on the whole contributes $5 + 16 = 21$ electrons. Thus there are 13 atoms to 21 electrons.

The same rule holds in γ-brass, which is represented by the formula

[1] "The Structure of γ-Brass." Proc. Roy. Soc. (A), vol. cxii (1926), p. 678.

Cu_5Zn_8. Since zinc is of double valency the ratio 13 to 21 again holds. There is a similar structure in the copper-aluminium alloy represented by Cu_9Al_4. Each aluminium atom contributes three electrons, and again the ratio of atoms to electrons is 13 to 21. In the copper-tin series the chemical formula of the corresponding phase is $Cu_{31}Sn_8$; each tin atom contributes four electrons, and so the ratio is 39 to 63, that is again 13 to 21. What this means is that the structure of the alloy and its properties are governed, not by the proportion between the numbers of the atoms, but by that between the numbers of the electrons and the atoms, the two (or it may be more) sorts of atoms being lumped together. This very remarkable law, which holds in many other cases, is known by the name of Hume-Rothery's law. The picture of an alloy which the law presents is entirely new and obviously most important. It is clear that our knowledge of alloy structures is undergoing extensive development as we learn more and more of the crystalline state.

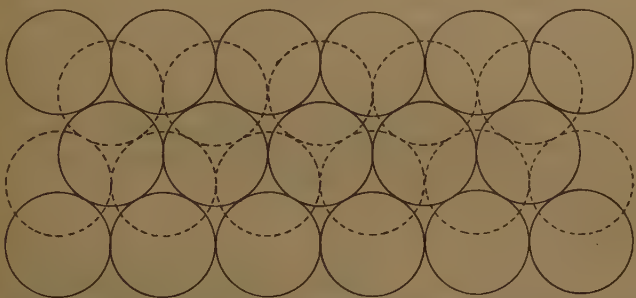
All that I have said so far relates to the crystalline structure of metals and alloys. I have tried to explain that by means of the X-ray methods we can enumerate the various phases of combination of two or more metals into an alloy, and can find the structural arrangement of the atoms in each phase. The work so far is descriptive, and no more. But it opens the way to a wide field of inquiry in which we try to find the relations between the description of a substance and its properties, physical, chemical, and so on.

Let us remind ourselves that the X-rays tell us the arrangement of the atoms in the metal, giving us their dimensions and relative dispositions; in fact, furnishing us with what might be called a "working drawing." Some metals, for example gold, silver, copper, and aluminium, can be described in simple terms as groups of spherical atoms packed as closely as possible. The atoms may be considered to be positively charged, and to be held together by the attractions of negatively-charged electrons which act as a kind of cement. Such a view is temporarily satisfactory, although it needs much amendment and enlargement when the question is considered in greater detail. Of close-packed structures composed of spheres, there are two kinds, one of which gives a cubic form to the crystal, and the other a hexagonal form. The metals mentioned above take the cubic form; magnesium and beryllium are examples of hexagonal crystals. Iron between 1,100° C. and 1,425° C. has the former structure. Above and below this range the structure is quite different. Each atom has now eight neighbours lying at the corners of a cube, itself lying at the centre. It has what is called the body-centered form, of which the β-form of *Fig. 4* (facing p. 185) may be taken as an illustration when all the atoms in the figure are

supposed to be alike. Some metals are not quite so simple, as, for example, cadmium which has the η-structure of *Figs. 5* (p. 185).

With such models before us we can understand at once the various features of the external form of the crystal and the fact that the faces make definite angles with each other, although we may well be surprised at the extraordinary uniformity in the values of these angles displayed by the particular crystal, no matter what its source may be. We can appreciate the special forms of the etch figures on different faces, and, if the crystal is not isotropic, we can observe how its expansion with rise of temperature is related in amount and direction to the atomic arrangement. We can note the planes on which the crystal tends to slip when shearing stresses are applied. It is found that in general these are the planes of closest packing; that is to say, of all the ways in which the regular array can be

Fig. 6.



A Set of Spheres Lying in a Plane (Full Lines) may Slip on a Set of Spheres (Dotted Lines) Lying in a Neighbouring Plane. Any Line Joining the Centres of Two Spheres that Touch Each Other may be a Direction of Slip.

considered as a set of parallel sheets, that which puts the greatest number of atoms into a unit of area of the sheet, defines the plane on which slip tends to take place. Moreover, it appears that the direction of closest packing in that sheet is the direction in which slip takes place. The model shown in *Fig. 6* will help to make the point more clear, and will show how the observed facts are in accord with what might be expected as a consequence of structure.

So far, so good: but we are obviously far from the explanation of all that we observe. The differences between crystals of gold, silver, aluminium, iron, and so forth are more than structure can account for. We ascribe these, however, to chemical considerations. A newer and very curious problem presents itself when we observe that the same metal may have widely different properties while its crystalline pattern remains unaltered. For example, a single crystal of copper can be bent with the fingers; motion along the

slip planes is easy to bring about. But if the crystal has been bent once or twice, it becomes much more difficult to bend again. To use the ordinary technical term, it has been "hardened by cold working." In fact, the hardness, the rigidity, and the strength of the crystal can be greatly varied although the X-rays show no change in the pattern. The X-rays do, however, show what can often be observed by the naked eye, that the single crystal has been broken up into separate "grains." There is a never-ending surprise in the fact that when the continuity of the single crystal has been broken, it is stronger than before.

Plasticity, toughness, hardness, and some other properties are known as "structure-sensitive" properties, as compared with lattice-dimensions, refractivity, density, and other "structure-insensitive" properties. The structure here referred to is the granular constitution, not the lattice of the crystal. There might be confusion between the two were it not that the terms are well established.

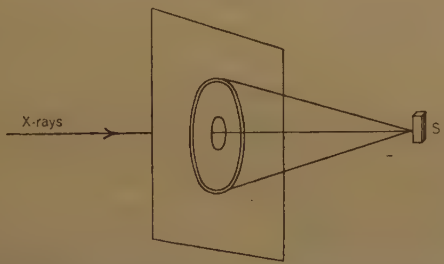
The "structure-sensitive" properties of metals are extremely important in all constructive work. It is the task of the metallurgist to control them, and the hard-won knowledge and skill of the metal-worker are based on the discovery of the treatment which in any one case gives the result that is desired. Heat-treatment, forging, alloying—sometimes with mere traces of foreign substance, sometimes in bolder mixtures—all these in an infinite variety of ways modify the worker's result, and must be studied as far as is possible. The X-ray methods open up a new line of attack by which it may be possible to obtain clearer views of the problem. Much work has already been done, and more is now in progress. The problem is so complicated that it would be absurd to expect a far-reaching solution for some time to come. Nevertheless some very interesting results have already been reached, and the promise of further deductions is great. Perhaps the best way of giving some idea of the nature of the attack on the problem, and of the positions reached, is to consider very briefly a typical investigation. Let us take as an example the work on fatigue of a mild steel carried out at the National Physical Laboratory by Dr. H. J. Gough and his colleagues.

In this research the X-ray diffraction photograph of the specimen is obtained in the manner shown in *Fig. 7*. The fine pencil of X-rays passes through a hole cut in the photographic film and strikes the specimen at S. Diffracted X-rays issue from S: some would go forwards and would form rings on a film prepared for their reception if the specimen were so small that the rays could get through it, but in this case the specimen is too thick. Other diffracted rays are returned at larger angles and can fall on the film as shown in *Fig. 7*, making rings concentric with the original pencil of X-rays. Examples are shown in *Figs. 8* (facing p. 192). The sizes of the rings depend on

the nature of the specimen, and on the wave-length of the X-rays. For the present purpose, however, size is of no importance, as the research is concerned not with the nature of the specimen but with its condition. The latter is determined by an examination of any one of the rings. In *Figs. 8* the ring is formed of a number of spots, distributed irregularly. Each spot is due to a single crystal: clearly the specimen consists of a number of small crystal-grains. The completeness of the circle is due to the fact that there are grains oriented in all possible ways.

The specimen is now subjected to stress, static or cyclic. If the stress is large enough the appearance of the ring changes. The photograph, *Figs. 8* (*a*), was taken when a static stress of 14 tons per square inch had been applied, up to which point the picture agreed with the picture obtained before any stress had been applied. Each

Fig. 7.



The Experimental Arrangement by which the Photographs Shown in *Figs. 8* (Facing p. 192) were Obtained.

spot is so far clear and distinct, implying that the crystal grains have so far been unaffected. But the appearance changes when the stress is increased to 16 tons per square inch, and the specimen begins to yield (*Figs. 8* (*b*)). Some of the spots begin to spread along the circumference of the circle. This means that the corresponding grains are breaking up into smaller grains, which do not make any large angles with one another since the spread is only small. The relative orientations are all included in about 2 degrees. Also, and this is a distinct effect, a faint continuous haze begins to form all round the circle. It can only be due to minute crystals, the debris, it might be said, of the breaking up of the original grains. A fairly close estimate can be made of the size of these fragments. The absence of any distinguishable spots in the haze gives an upper limit of about from 10^{-4} cm. to 10^{-5} cm. If they were smaller than this the ring would spread radially and become hazier. The principle is the same as that which holds in optical diffraction. If there are too few lines in a diffraction-grating, the spectral lines lose their sharpness and

become blurred. Similarly, in the X-ray spectrum, if the diffracting crystal is too small the ring loses definition, and this sets a lower limit to the size of the fragments. Since this size is defined with some strictness, and since the fragments begin to appear as soon as the grains break up, and since also it is found to be very difficult to reduce the size of the fragments still further, it would seem that we have here objects to which a name may reasonably be given. It is usual to speak of them as " crystallites."

The stress is now increased considerably above that at which the specimen yields (*Figs. 8* (*c*)). The original grains are still further broken up, and the amount of crystallite has grown considerably, since the haze now runs continuously around the ring. Finally, in *Figs. 8* (*d*) the stress has caused fracture. Nothing is left but a mass of crystallites, oriented in all possible ways.

When cyclic stresses are applied, the same changes in the crystalline condition are found to take place.

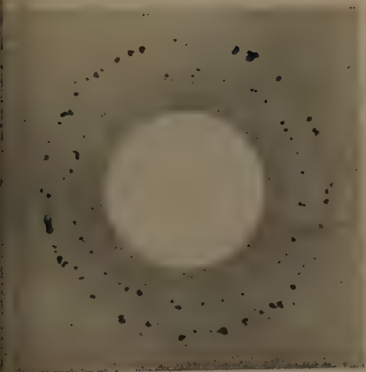
From observations such as these Messrs. Gough and Wood have been able to draw important conclusions. Suppose that a stress is applied which is more than enough to cause yielding but is not enough to cause fracture. " As a result, crystal break up into a smaller grain size takes place, together with the production of crystallites. The specimen may then be cyclically unloaded and reloaded to and from a lower stress. . . . After a very considerable number of such stress-cycles has been applied, an examination will show practically no further deterioration in structure, the state of grain dislocation and the number of crystallites produced remaining essentially as after the first application of the superior stress of the cycle." [1] " . . . the main conclusion is that the application of cycles of a safe range of stress (strain) is unable to cause *progressive* damage to the state of the structure, a stable state has been set up." [2]

I take this research by Messrs. Gough and Wood as an example of the kind of work which is going on in many laboratories, prompted by the facility with which the composition, orientation, and size of the crystalline grains can be determined by the X-ray methods. Much of the work leads to, and indeed is primarily concerned with, the conception of the " crystallite." Some writers have supposed that, over and above the crystalline structure which is detected by the X-rays, there is a regular superstructure of much larger mesh dimensions, whilst others have spoken of " groups," or of " lineages," that is to say, branches which during solidification from the melt radiate from the initial nuclei of growth, sub-dividing into lesser

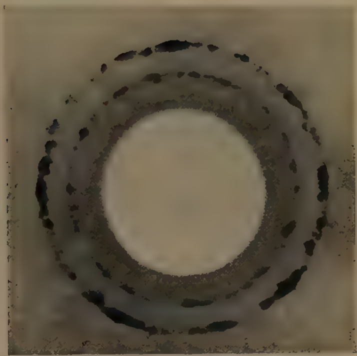
[1] " A New Attack upon the Problem of Fatigue of Metals, using X-Ray Methods of Precision." Proc. Roy. Soc. (A), vol. 154 (1936), p. 537.

[2] *Ibid.*, p. 538.

Figs. 8.



(*a*) 14 tons per square inch: below yield-point.



(*b*) 16 tons per square inch: at yield-point.

(*c*) 22 tons per square inch: above yield-point.

(*d*) After fracture at 27·2 tons per square inch.

PHOTOGRAPHS SHOWING CHANGES IN STRUCTURE DUE TO STATIC STRESS, WHICH ARE OF THE SAME TYPE AS THOSE DUE TO CYCLES OF STRESS OF RANGES LEADING TO FRACTURE.

Figs. 9.

(*a*) Al foil thinned in dilute KOH: relatively large random crystals.

(*b*) Crystal-growth and orientation produced by heating.

(*c*) After further heating, one large Al crystal with (100) orientation has been formed.

(*d*) With more drastic heating Al_2O_3 is formed, though part of the Al crystal still remains.

CRYSTAL-GROWTH AND ORIENTATION PRODUCED BY HEATING.

"lineages" until the whole volume of the crystal is solid. There is no doubt that in the metal there is something more than the regular arrangement revealed by X-rays. The complexity and variety of the physical properties which depend on the history and the treatment of the metal are far more than can be accounted for by the simple picture of a group of positively-charged spheres of uniform size, held together by negatively-charged electrons. From a number of different sources it would appear that the crystallites are fairly well-defined in size, having linear dimensions of about 10^{-4} cm. If this is so, there must be some atomic property at work other than the form and dimensions of the sphere, because a number of equal spheres have but one term of definition, namely, the radius, and there is no basis on which to build a second geometrical definition, which is to be the size of the crystallite. Impurities might cause an internal separation into groups, but if that were the origin of the crystallites their size would surely depend on the amount of the impurity. Actually, there is experimental evidence that this is not so; for instance, the spacing of glide planes in lead crystals,[1] which is surely associated with any natural sub-division, has been shown to be independent of a range of factors, and therefore to have a physical significance.

It is an interesting fact that the perfect crystal is a rarity. Diamond and Iceland spar are nearly perfect, but rock-salt was found at an early date of the X-ray researches to be very imperfect. Optically, it seems clear and whole, but the X-rays find it to be a mosaic of smaller crystals. It is nearer perfection when it is formed from the melt; as it cools, especially if it is worked, it takes on the mosaic condition, and it also does so if it is formed by evaporation. Similarly, a crystal of sodium formed from the liquid at about 95° C. is a much better crystal than when it is cooled down. It has recently been shown by Mr. R. Dawton at the Royal Institution that a certain re-forming takes place during reheating, and that there is a hysteresis effect if the temperature is varied in cyclic fashion. If, however, the crystal is much cooled, as in liquid air, there is a more thorough break-up, and afterwards no hysteresis effects are to be found. The original condition is only to be restored by melting.

It would seem that the perfect crystal is not, so to speak, the natural form; in other words, it is not the form of least energy. The atom may be treated as isotropic when the temperature is high, but at a lower temperature when its motion is less violent and its effects in different directions are not subject to averaging, the ideal form of

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[1] E. N. da C. Andrade and R. Roscoe, "Glide in Metal Single Crystals." Proc. Phys. Soc., vol. 49 (1937), p. 152. (March, 1937.)

arrangement is no longer adhered to, although the departure therefrom is insufficient for detection by X-rays. One might say that the pattern can only be adhered to strictly by a restricted number of atoms.

Whatever may be the final aspect in which the crystallite must be regarded, it is clear that the condition which it represents more or less perfectly is fundamental to our understanding of metallic properties, and especially those which most affect the work of the engineer. For that reason the present situation is of great interest, and work in this region is worthy of strong encouragement and support.

In the interior of a crystal each atom is held in its place by forces due to surrounding atoms. Near the surface the surrounding is incomplete, and the atoms may be expected to break away more or less from the normal pattern. There is, in fact, a surface layer in which the laws of the interior are modified. It is extremely thin, and its presence has no effect on the mechanical, electric and magnetic properties of the crystal as a whole. But it modifies very greatly such properties as depend upon the action of the surface atoms on other atoms brought near to them. Such effects are considerable and often very important. All those included under the heading of catalytic action are of that kind. Corrosion, for example, depends on surface arrangement. When the oxygen atoms settle on the surface of a metal and form oxides, the readiness with which the effect occurs varies not only with the metal itself, but also with the state of the surface. Lubrication-effects are also connected with surface arrangement. In matters of biochemistry surface actions are of the greatest importance, since physiological processes depend on the nature of the surfaces at which they occur. It must not be thought that the internal crystalline arrangement is therefore of no importance; on the contrary, the surface conditions are derived from those inside.

The X-ray methods cannot give information about surface conditions because their penetration is too great, and their effects are determined by the state of the interior. Consequently the still more recent methods of electron-diffraction have come as a most welcome reinforcement. The very remarkable discovery that X-rays and electrons are to be regarded as interchangeable forms, that both have corpuscular properties and wave properties, and differ rather in degree than in kind, led naturally to the attempt to use electrons for the exploration of crystalline structure in the way that had been successful with the X-rays. The electron stream of an ordinary electric discharge in an exhausted chamber is equivalent to a train of very non-penetrating X-rays: it is diffracted in the same way,

but all the diffraction occurs in the first few atomic layers of the solid. In a remarkably short space of time—a few seconds generally—an electron diffraction photograph is formed by the surface layers, the interpretation of which is of the same general character as that of the X-ray photograph. Only in certain details of appearance and interpretation are there real differences. A few pictures will serve as illustrations (*Figs. 9*, facing p. 193).

Certain very interesting conclusions are drawn from pictures such as these. There is, for example, new light on the nature of a polished surface. The late Sir George Beilby showed that in certain cases the polished substance "flowed"; scratches and flaws were covered over by what he supposed to be an amorphous layer. If the layer were removed by etching, the original scratches might reappear. Such a polished body, when examined by X-rays, still shows the crystalline character of the body, but when the diffraction methods are used it appears that, in some cases, although by no means in all, the top layers are indeed non-crystalline. The properties of such a surface are quite different from those of the crystal. For example, Mr. G. I. Finch has shown that when zinc is sprayed on to the normal crystalline surface of copper, zinc crystals are formed, which indeed continue, as well as they may, the ordered arrangement which they find there. If, however, the surface has been polished, the zinc first deposited by the spray shows for a short interval that it has taken a crystalline form, and then it dissolves into the amorphous-copper layer as if it were a liquid. When the spraying is continued the absorption ceases after a while. It is as if the atoms of copper in the polished layer, not being properly bound up with other copper atoms, enter with especial ease into combination with the zinc.

The series of photographs shown in *Figs. 9* (facing p. 193) is due to Mr. G. I. Finch.[1]

They show another surface phenomenon of great interest. A thin foil of aluminium gives the electron picture of *Figs. 9* (*a*). The rings are nearly complete, but have a spotty character. This shows that the foil consists of a number of small crystals variously oriented. When the foil has been quickly drawn through the hot gases of a Bunsen flame the picture changes to that of *Figs. 9* (*b*): the crystals are fewer in number and larger in size. Further heating leads to the still greater regularity of *Figs. 9* (*c*); a single large crystal has formed, and the picture shows a rectangular arrangement of spots. More drastic heating brings about a chemical change (*Figs. 9* (*d*)). Aluminium has combined with oxygen to form crystals of corundum,

[1] G. I. Finch, A. G. Quarrell, and H. Wilman, "Electron Diffraction and Surface Structure." Trans. Faraday Soc., vol. xxxi (1935).

(Al_2O_3), the ring pattern being characteristic of that substance. The crystals are very fine, so that the rings are continuous. Since intense local heating is produced at a surface where rubbing is taking place, minute crystals of corundum must be formed where there is sufficient friction between aluminium and any other substance.

If now we turn from the consideration of metals and alloys to that of other substances, we must give first place to the earths, rocks, and clays which form the crust of the earth. These also are made use of in the constructions of the engineer. They may be used in their natural state, or they may be fired to form bricks, pottery, glasses or refractories. Here again, we find that at the bottom of all the phenomena lie on the one hand the tendency for atoms and molecules to arrange themselves in regular order, and on the other hand the opposing tendencies to disruption and irregularity, due to temperature, impurities, conditions during formation, and so on.

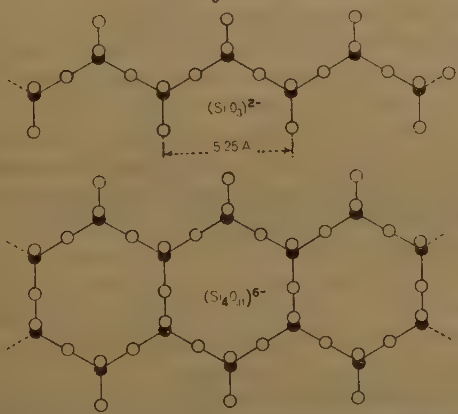
Oxygen and silicon are by far the most common constituents of the surface of the earth. Half the known world is composed of the former, and a quarter of it of the latter. It is not surprising, therefore, to find that these two elements form the basis of a great class of minerals known as the silicates. Of this class the simplest is quartz, which contains silicon and oxygen only, in the proportion of one of the former to two of the latter. Here we find a type of construction which is totally different from that of the metals. The atoms are held together by strong bonds, termed by the chemist "covalent." It is convenient to think of covalent bonds as due to a sharing of electrons by neighbouring atoms, each atom incorporating them into its own structure. The two contiguous atoms are thereby held together in far stronger fashion than is found in metals, where the link is indirect, and is due to the attraction of each atom towards a free electron. In a substance like quartz, where all the bonds are covalent, there are no free electrons; quartz is a non-conductor of electricity.

In all the silicates—and this is a very curious fact—every silicon atom lies at the centre of a regular tetrahedron formed of four oxygen atoms. A silicon atom makes its connection with another silicon atom in quartz by the inclusion of one and the same oxygen in the two corresponding tetrahedra (*Figs. 10*). It is clearly possible to form chains of tetrahedra in this way, and quartz may be defined as consisting of such chains placed side by side and linked together by oxygen-sharing. The chains are twisted to form spirals, which accords with the well-known capacity of quartz for rotating the plane of polarization of light. When quartz is sufficiently heated it becomes viscous. Bonds are broken but chains still exist of irregular form and length, based no doubt upon the spirals of the original

structure. The substance may now be likened to a mass of sticky intertwining fibres which once had lain side by side. There are still links to make the substance strong, but it has lost the properties which depended on the spiral arrangement. It is less tightly packed, and therefore the density is less. The heated substance is viscous because the chains, although linked together, are free to move over each other.

This is also the structure of glasses of various kinds. The irregular and irregularly-connected chain-like formations are in existence here also, but metal atoms, such as sodium, are interspersed here and there and form intervening links, weak compared to those of fused quartz. There is always a tendency for the chains to form up again

Figs. 10.



SILICON–OXYGEN GROUPING IN THE SILICATES : CHAINS AND BANDS.
(From *Zeit. für Krist.*, vol. 74 (1930), pp. 269 and 270).

as in quartz, so that a glass may in part recrystallize, or as it is termed, be "de-vitrified." Such changes are to be found in many other materials, and lead sometimes to important changes of properties. They instance the continual tendency to regularity of arrangement.

There are minerals in which the long chains of linked tetrahedra, each composed of four oxygen atoms at its corners and a silicon atom at the centre, lie side by side, joined together by connecting links of metal atoms. Asbestos belongs to this class, and because the metallic links—calcium and magnesium—are so much weaker than the oxygen-sharing bonds of the chains themselves the material is easily split up into fibres. Again, the oxygen tetrahedra may be linked together so as to form sheets, which are fastened together by

intervening atoms of metal. Mica may be taken as an example; the ease with which it can be split is due to the strength of the separate sheets and the relatively feeble nature of the metallic bonds that hold the sheets together.

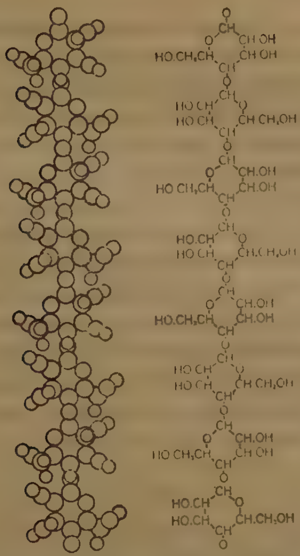
Aluminium atoms sometimes replace silicon atoms at the centres of the oxygen tetrahedra. Now aluminium is tervalent, whilst silicon is tetravalent. Consequently there is an unsatisfied valency for every tetrahedron; this defect is remedied by the addition of extra metal atoms, such as sodium, magnesium or calcium, in sufficient quantities. The structural plan of the crystal is unaltered, being governed by the skeletal arrangement of oxygens: this is the constitution of the felspars. In this class are also the zeolites. It is sometimes a simple matter to change the extra atoms, substituting sodium for calcium for example. Analcite, one of the zeolites, is often used for this reason as a water-softener.

After oxygen and silicon, the most common atom is aluminium, which also enters freely into association with oxygen, singly or in compounds containing other atoms. The simple compound, corresponding to SiO_2, is Al_2O_3, corundum, ruby, or sapphire. Sheets can be formed of aluminium and oxygen atoms in somewhat the same fashion as the sheets of silicon and oxygen already described, and these sheets alternating with the silicate sheets form the clays.[1] The bonds between the two types of sheet are of the hydroxyl (OH) type and are relatively weak, so that the layers slide easily over one another. The X-ray photographs show that as clay absorbs more and more water the sheets remain unchanged in structure, but are forced apart from one another by the intruding atoms.

Although the crystals that are of most direct interest to the engineer are of mineral origin, there are many of less immediate interest to be found among the organic substances of the animal and vegetable world. Natural fibres are woven into cords and textiles. In this region is found the curious insistence of Nature upon the long molecule formed in the fashion of a chain by adding together link after link of some standard pattern. A picture of the cellulose molecule is shown in *Figs. 11.* Each ring contains five carbon atoms and one oxygen atom, and with certain attachments forms one link of the chain. It is strange how economical Nature is in her selection of designs, for this cellulose pattern is universal in the vegetable world. Long chain molecules link themselves side by side to form in the fibre a regular arrangement which deserves to be called crystalline and gives X-ray and electron pictures. The tensile

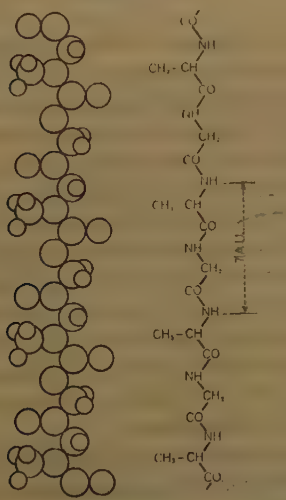
[1] C. E. Marshall, "The Constitution of the Clay Minerals." *Science Progress in the Twentieth Century*, vol. 30, p. 422.

Figs. 11.



THE ATOMIC ARRANGEMENT AND CONVENTIONAL FORMULA FOR PART OF THE CELLULOSE CHAIN.

Figs. 12.



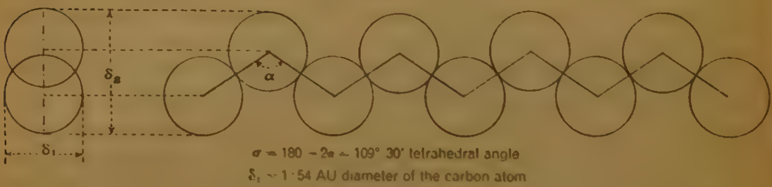
THE ATOMIC ARRANGEMENT AND CONVENTIONAL FORMULA OF PART OF THE FIBROIN CHAIN OF NATURAL SILK.

strength and flexibility of the chain are the basis of the similar properties of the fibre.

The protein chain is equally important to animal life. A picture of one of its simplest forms is shown in *Figs. 12* (p. 199). Its chief characteristic is the backbone formed of a series of carbon and nitrogen atoms, two of the former to one of the latter. Side chains of many kinds can be attached to it, hence the vast variety of the protein family. Here also there is often sufficient arrangement to give crystalline character. At the present time biochemists are particularly interested in certain forms of protein known as vitamins and viruses, and their crystalline character is important. Various forms of protein are the chief constituents of hair, wool, and horn.

Much attention is paid in these days to substances in which units of regular shape are linked together in irregular fashion. One of the best known and most used is "bakelite," formed by the action of formaldehyde on derivatives of benzene such as phenol. The regularly shaped hexagons of the benzene ring are tied together

Fig. 13.



corner to corner in irregular fashion, forming a three-dimensional network of great strength, a non-conductor of electricity. The class of resins to which bakelite belongs has been much enlarged of late.

Another long-chain molecule of great interest and importance is the zig-zag chain of carbon atoms which is the basis of all the paraffins, fatty acids, and alcohols (*Fig. 13*). When it is of sufficient length it holds on to a neighbour lying beside it with such force that at ordinary temperatures the substance is solid. A number of molecules arranged like the stalks of corn in a field form a cohesive layer. In the paraffins the stalks are upright, in the fatty acids they are on the slant as if blown over by the wind. In the solid substance layer lies upon layer, but the force between the layers is relatively weak so that one slides easily over the other. Hence arises the value of some of these substances as lubricants.

To sum up, the origin of the properties of all the substances that we use in our constructions is to be considered in relation to the mode of assemblage of the atoms of which they are composed. In nearly all solid bodies there is pattern involving regularity of arrangement;

that is to say, the substance is to that extent crystalline. The crystallinity is largely responsible for the characteristics of the body in which it is found, and since we can investigate it as it occurs both within the substance and on its surface, we find ourselves in possession of a new and powerful means of examining the materials with which we build. We may devise new materials, we may learn how to improve those that we have already, or to use them better, and we have a new insight into the wonderful and fascinating construction of the world animate and inanimate.

The situation being as I have tried to describe it, it is not surprising that the crystalline character of materials is under investigation in laboratories in many countries, both in devotion to the cause of the increase of knowledge and on behalf of the more immediate applications to industry. We in Great Britain are trying to do our part. At the National Physical Laboratory Dr. Gough and his colleagues have under way a research on the fatigue and other properties of metals to which I have already made some reference. Also, at the same Laboratory, there is a small section of the Physics Department which, under Dr. Shearer, is at work on instances of direct application of the new methods to industrial problems, including the cold-working of metals, corrosion, jewel-setting, paints, chromium deposits, transformer steels, and so on. A pamphlet issued by the Department of Scientific and Industrial Research gives a short account of this work. At Manchester University great progress has been made with the examination of the structure of alloys. At Cambridge the work is of a general kind, but has lately been carried on in collaboration with the biochemists, with most interesting results. In Leeds the crystal properties of textile fibres have been considered, and more recently the corresponding properties of nerve and muscle to which the former study leads quite naturally. At the Royal Institution particular attention is given to the organic substances, such as the paraffins and fatty acids, and to the accurate determination by a new method of the form and size of various organic molecules. Good work is being done at Birmingham and Bristol Universities. As one civil engineer (by your grace) to other civil engineers I commend all these efforts to you, hoping that my Lecture may have added to your interest in them, and begging you to help them forward whenever the opportunity presents itself.

Figs. 1, 2, 3, 4, 5, and *10* are reproduced by permission of Messrs. G. Bell and Sons; *Figs. 8* and *13* by permission of the Royal Society; *Figs. 9* by permission of the Faraday Society; and *Figs. 11* and *12* by permission of the Oxford University Press.

Sir Clement Hindley, Vice-President, in moving a vote of thanks to the Lecturer, said that there was no greater intellectual pleasure than to be led along through unaccustomed paths to new methods of thought; and to engineers he thought it was a particular pleasure to be taken along first of all through paths that were to some extent familiar and then through more and more unfamiliar paths to a new wonderland of science which, although it might please and amuse and interest them, had, as Sir William Bragg had so clearly brought before them, a very practical bearing on their work. Engineers were accustomed to dealing with Nature in its more massive forms, and, therefore, it must have come as a kind of revelation to many of them to find that those great masses of steel, concrete, and timber with which they had to deal were really built up of the ultra-microscopic wonders which Sir William Bragg had shown them; and that the internal structure which was at present being explored was the real basis of many of the physical properties which gave engineers so much anxiety and so much trouble and, in many cases, so many triumphs. There was no scientist of modern times more capable than Sir William Bragg of taking one gently by the hand into those new worlds of scientific thought, and they ought to feel very grateful to their Lecturer for having brought them that new knowledge. It was by way of giving them the facility of a new eye, or indeed more than one new eye, into the nature of things. The fact that the X-rays and the electrons could be made to give a kind of signature for every different material, equivalent in fact to human finger-prints, no two of which were alike, was very stimulating to the imagination and of the greatest value to engineers in their work. They were particularly grateful to Sir William Bragg for saying, as he did, that he felt he might talk to them as an engineer. There was no greater compliment that he could pay to them. He had therefore great pleasure in moving a vote of thanks to Sir William Bragg for what he thought would be considered one of the greatest of the James Forrest Lectures.

Dr. R. E. Stradling, in seconding the vote of thanks, remarked that he would like to refer to one other side of Sir William Bragg's work, and that was his sustained interest in the application of science. Knowing from his own work and by his own contacts with Sir William Bragg, who came to them as the President of the Royal Society and as the leader of pure science in England, he could testify to the very close contact Sir William Bragg, had made in the application of science to industry which The Institution represented. For that and for the magnificent Lecture which he had given, the thanks of the Members were due to Sir William Bragg.

JOINT MEETING WITH THE INSTITUTION OF STRUCTURAL ENGINEERS.

18 March, 1937.

LT.-COL. C. H. FOX, O.B.E., B.Sc., F.S.I., President Inst. Struct. E., in the Chair.

"New German Bridges."

By Dr.-Ing. G. Schaper, Director of the German State Railways.

Abridged Report.[1]

High Tensile Steel.

Dr.-Ing. Schaper said that, whilst German bridge engineering could not point to achievements on the monumental scale of the Forth bridge, it had been characterized by an endeavour to make progress in the development of new materials and in the application of new methods of construction, reconciling the demands of pleasing architecture with the principles of statics and the canons of sound construction. The great increase that had taken place in the axle-loads of locomotives and rolling stock on the German State Railways was the occasion for interesting the steelworks in a successful endeavour to produce a steel of higher grade (St. 52) than the usual St. 37 grade, which would enable the design of large steel bridges to be improved and the cost to be reduced. The specifications of the German State Railways laid down the following values for St. 52 :—

For a rolling thickness up to 0·71 inch a breaking stress of 33·0 to 39·4 tons per square inch, an elastic limit of at least 22·8 tons per square inch, an elongation at fracture of at least 20 per cent. in the direction of rolling and of at least 18 per cent. transversely.

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[1] Published in full in Journal Inst. Struct. E., vol. xv (1937), p. 209. (May, 1937.)

For a rolling thickness of 0·71 to 1·18 inch a breaking stress of 33·0 to 40·6 tons per square inch, an elastic limit of at least 22·2 tons per square inch, and an elongation of at least 19 per cent. in the direction of rolling and of at least 17 per cent. transversely.

For a rolling thickness in excess of 1·18 inch, a breaking stress of 33·0 to 40·6 tons per square inch, an elastic limit of at least 21·5 tons per square inch, and minimum elongation 18 per cent. longitudinally and 16 per cent. transversely.

The permissible stresses for St. 52 were, in general, 50 per cent. higher than those for St. 37, but like all high-tensile steels (and also nickel steel) St. 52 suffered from a disadvantage in regard to fatigue resistance when used for making riveted connexions, or drilled for rivet holes; thus, when subjected to a stress which alternated between zero and some upper limit, its fatigue resistance was not much greater than the resistance of St. 37 to repeated non-alternating stress. If, however, the dead load which pre-stressed the member were increased, there was no appreciable reduction in the amplitude of oscillations corresponding to a given fatigue resistance against pulsating stress, and the fatigue limit approximated to the elastic limit. For members carrying a heavy dead load, therefore, St. 52 was superior to St. 37, because the elastic limit of the former was so much higher.

In the case of road bridges the question scarcely arose, because the live loads assumed as a basis for the design would hardly ever occur in practice in their most unfavourable combinations, and fatigue stresses were consequently of no practical importance; hence, in those bridges the high strength properties of St. 52 could be utilized without reserve. In the case of railway bridges under intense traffic, however, the loading assumed for the calculations might frequently occur in actual practice, and considerations of fatigue might become important. In the smaller bridges of that category the dead weight of the members was low in relation to the live-load stresses and St. 52 was not, therefore, a suitable material to use. In the case of larger railway bridges, however, the majority of the members were subject to heavy dead loads, and it was only in the remaining few, wherein the dead-load stresses were low, that the permissible stress would need to be correspondingly reduced.

Thus, in the smaller railway bridges, as in the smaller road bridges, the use of St. 52 was ruled out on economic grounds, and St. 52, being dearer weight for weight than St. 37, became economical only in bridges of larger span which offered scope for a considerable

saving in weight through its use. St. 52 was therefore the proper steel to use in the construction of major railway and road bridges, and in those the saving in cost by comparison with St. 37 might be as much as 15 per cent.

In order to safeguard the welding qualities of St. 52 under all conditions (that was to say, in order to exclude any tendency to brittleness and cracking when welded) its chemical composition had been made subject to the following requirements :—

C content not to exceed 0·20 per cent.
Si " " " 0·50 " "
Mn " " " 1·20 " "
Cu " " " 0·55 " "
Either 0·30 per cent. Mn, 0·40 per cent. Cu, or 0·20 per cent. Mo might be allowed in addition.
S and P should not exceed 0·06 per cent. each or 0·10 per cent. together.

Design of Girders.

The present practice in Germany was to prefer solid-webbed to open-webbed ginders whenever an economic case could be made out for so doing. Plate-web girders had been built for road bridges up to 360·9 feet span and for railway bridges up to 177·2 feet span, their advantages being ease of maintenance and pleasing appearance. Plate-web girders were also being preferred in the case of arched and rigid-framed bridges.

Where trussed girders were adopted it was thought preferable, on æsthetic grounds, to make the upper and lower booms parallel rather than to make the upper boom curved. In bridges which were symmetrical about the centre of the stream, and in which the lengths of span decreased progressively towards the two banks, a favourite design was the "stepped parallel" elevation in which the ends of the upper booms of the deeper girders (spanning the wider openings) were connected by inclined posts to the upper booms of the shallower girders immediately adjoining—an arrangement which conferred on the bridge a unified and homogeneous aspect.

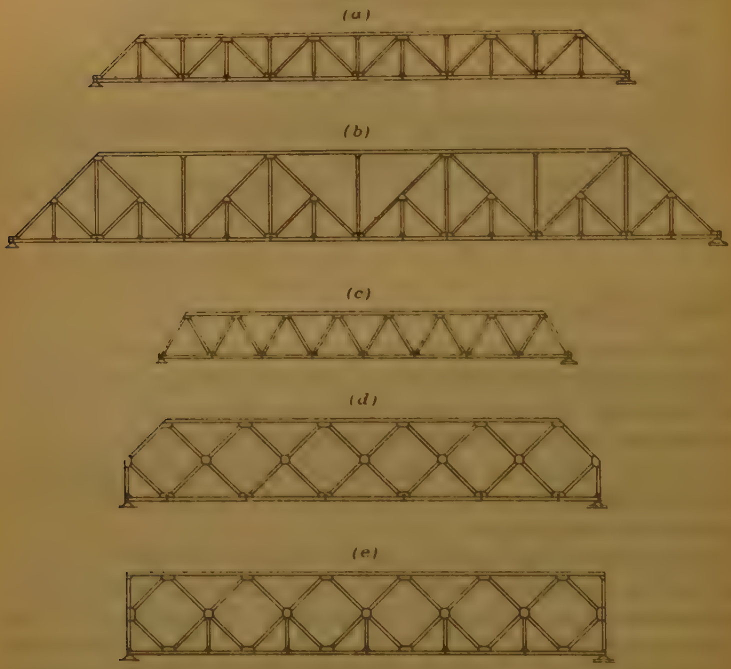
The forms of triangulation customary in the construction of trussed girders were the following, as illustrated successively in *Figs. 1* (p. 206) :—

(*a*) Truss composed of diagonals and verticals.
(*b*) Same as (*a*), with the panels subdivided.
(*c*) Truss composed of diagonals without verticals.

(*d*) Truss formed of crossed diagonals without verticals, the cross-girders being attached to the lower panel points.
(*e*) Same as (*d*), but with the cross-girders suspended from the intersection points of the diagonals.

A new type was a bridge of triangular cross-section having two lower booms but only a single upper boom. It offered the advantages

Figs. 1.



of marked economy for large spans and of great rigidity against torsional forces, whilst all upper wind-bracing and cross-bracing were eliminated.

Æsthetics.

Dr.-Ing. Schaper pointed out that the aspect of bridge structures should be simple and straightforward so that the general appearance was uniform and unmarred by sudden breaks. It was important, moreover, that such structures should merge smoothly into their surroundings, both those near and those at a distance. The abut-

ments should be designed to mark the ends of the work in a fitting way and to give a smooth transition into the neighbouring landscape. The form of the superstructures had to be properly related to that of the substructures, whether piers, trestles, or abutments.

In his Paper he gave various examples of work recently carried out and stated that in designing bridge structures there should be no architectural trimmings in the way of turrets, towers, pilasters, and the like, but the effect should depend solely on the beauty inherent in the structural forms, on proper correlation between superstructure and substructure, and on harmony with the surroundings. In Germany there had recently been instances, such as that of the Rhine bridge at Mainz, where an existing bridge cluttered with ugly and unnecessary architectural accretions had been freed from those and been converted, most strikingly, to a thing of engineering beauty.

Welding.

He said that the successful results obtained through the adoption of welding practice in mechanical engineering, rolling-stock construction, and shipbuilding gave rise to the question, some 10 years ago, whether technical and economic advantages might not also be obtainable by the same means in bridge work. After exhaustive preliminary experiments and theoretical investigation, the first all-welded railway bridge having plate-webbed main girders of 32·8-foot span had been completed in 1930. It had been thoroughly tried for more than 6 years, first on a branch and then on a main line, and in that period had been crossed by some 230,000 trains, subjecting the longitudinal girders and cross-girders to the loading of more than one million heavy locomotive axles in addition to more than ten million lighter axles of rolling stock. The bridge was then taken out of service for the purpose of detailed investigation. Up to the present no defect had been found in it by eye or with the magnifying glass; it was to be subjected to laboratory tests by radiography and measurements of electrical flow, and to fatigue experiments, extending to every detail and to every possible kind of defect.

Since that first welded railway-bridge was built extensive fatigue tests had been carried out on welded connexions and on completely welded girders, and, as the result of those, knowledge of the correct design of welded bridge-work had been so enlarged that even bridges of long span could be constructed in that way with perfect safety. There were, in fact, already one hundred and fifty welded railway-bridges with plate-webbed main girders in service on the German State Railways, the maximum span being 177·2 feet. A further

number of bridges of that type had been constructed for the Reichsautobahnen (special motor roads) with spans up to 337·9 feet.

In Germany, up to the present, the only welded railway and roadway bridges had been solid-webbed girders. The use of welding in the construction of trussed railway-bridges was still exposed to challenge as the question of how to ensure perfect connexions at the intersection points, possessing the requisite high degree of fatigue resistance, had not yet been cleared up. Trussed bridges for highways could, however, safely be built by means of welding, as in those the consideration of fatigue effects was of minor importance.

Shrinkage Stresses.

The welding of plate-web girders entailed the development of considerable shrinkage stresses in the longitudinal seams connecting the flanges to the web, amounting to as much as 12·7 tons per square inch, and many engineers had feared that those high stresses, in augmentation of the ordinary dead-load and live-load stresses, were bound to lead to the early failure of the girder. They had, however, overlooked the fact that rolled joists likewise acquired very heavy residual stresses from the rolling process but, nevertheless, carried as heavy loads as girders which were free from all internal stress. It had, moreover, been conclusively demonstrated by means of numerous tests that welded plate-web girders possessed greater fatigue resistance than riveted girders.

Shrinkage stresses, then, were not a source of danger, and they played no part in determining the load-bearing capacity of a girder. One reason for that was that heavily stressed parts were surrounded by parts carrying much lighter stresses, but a more fundamental reason was that the shrinkage stresses were from closed systems which were in equilibrium on their own account and which were not superimposed on the normal dead-load and live-load stresses of the structure.

In welded portal-frames the shrinkage stresses resulting from the welding of the web plates at the corners attained values of the order of from 12·7 to 16·5 tons per square inch. It had further been ascertained by means of fatigue tests that joints in the web and flange plates of solid-webbed girders were most effectively made in the form of simple butt welds, double-V, V, or U-shaped in cross-section. The joints were not improved by the addition of cover straps. It was essential, of course, that the seams should be completely free from defects, and that there should be no incipient cracks in the successive runs of weld-metal. The freedom of the butt welds from defects had to be checked by X-ray photographs. It

was best to form the butt joints in the flanges at an angle of 45 degrees to the longitudinal axis of the girder as they would then possess considerably greater fatigue resistance than if arranged at right angles. The transitions between the butt welds and the parent material should be milled or ground to combat the notching effect, as in that way the fatigue resistance of the connexion was greatly increased.

It might, then, be considered as proved that shrinkage stresses were not a source of danger in the girders themselves. Suitable precautions should, however, be taken when making the butt welds in the joints of the longitudinal girders, cross-girders, and wind-bracings to ensure that such stresses were kept down to a minimum. With that object in view the procedure followed in assembling the deck of the bridge was as follows: first, a single cross-girder was welded to the two main girders at the centre of the span; then it was welded to the ends of the rail bearers in the spaces on either side of it, and the far ends of those were welded to the cross-girders next in order on either side of the centre; next, those cross-girders which had hitherto been loose were welded to the main girders, care being taken to stress them in opposition to the shrinkage stresses during the process. That sequence was repeated, working from the centre towards both ends of the bridge, until the last cross-girder but one had been connected near each end. The endmost cross-girders of all had then to be welded accurately into place at the points provided for them, and the rail bearers in the end spaces were cut accurately to length on the site and were finally welded into position (between the last cross-girder and the last but one) after that had been done. If that procedure were followed the shrinkage stresses arising in the connecting seams of the rail bearers were limited to very small amounts.

Flanges for Plate Girders.

The types of flange construction used in welded plate-webbed girders are shown in *Figs. 2* (p. 210):

(*a*) An ordinary flat rolled plate welded to the ordinary square edge of the web plate.

(*b*) An ordinary flat rolled plate welded to a double-bevelled (V-shaped) edge on the web plate.

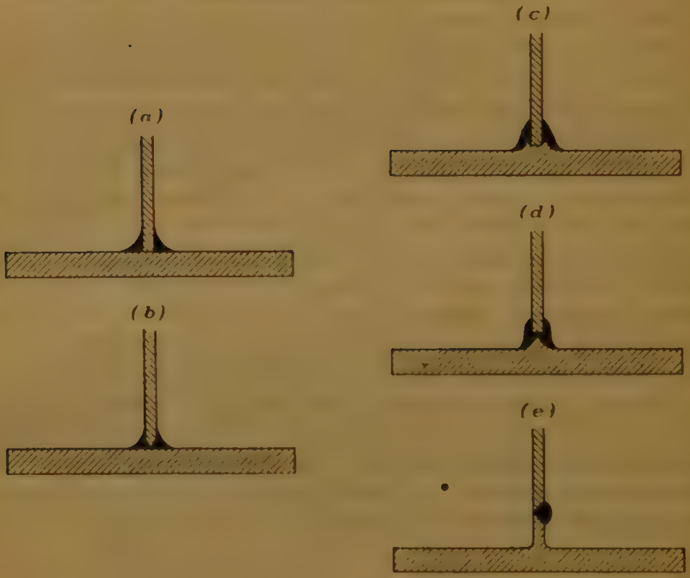
(*c*) A "nosed section" rolled with a protrusion along the centre line and a groove in the middle of the protrusion to receive the web plate.

(*d*) A "bulged section" having a central triangular protrusion which is welded to the web plate by two V-seams.

(*e*) A special form of T-section allowing the connexion to the web plate to be made by a single V-seam.

Where ordinary flat plates were used it was not possible to examine the seams connecting the web to the flange by means of X-rays, but that could readily be done when using the other kinds of section.

Figs. 2.



TYPES OF WELDED CONNEXION BETWEEN FLANGE AND WEB PLATE.

Welding Aids.

The simplest and most reliable welding was done with the seams in a horizontal position and with the centre-line of the cross-section of the seam vertical, and so the girder about to be welded was supported between plate-webbed circular sheaves fitted with rims whereon it could be rolled to bring each of the various seams into the proper position. Where the flanges consisted of ordinary flat plates the connecting seams between those and the web plate were

welded with the girder in an inclined position. The sheaves might be arranged either to roll forward and back on horizontal runners laid on the ground, or to revolve on fixed rollers. On many constructional sites it was impossible to make use of any turning mechanism, and vertical, or even overhead, welds might have to be carried out, such as were usually necessary in the course of erecting the decking of the bridge. If skilled and experienced welders were employed such work could be done with entire freedom from defects.

Advantages of Welding.

The main points that weighed in favour of the adoption of welding might be summarized as follows :—

(1) Welded structures were considerably lighter than riveted ones because, in the case of members subject to tension, no addition had to be made to the cross-section of a member to compensate for its reduction by rivet holes, and also because members meeting at 90 degrees or at any other angle could be connected directly by weld seams without the necessity for angle cleats as in riveted work. As a result, welded structures were up to 23 per cent. lighter in weight than their riveted equivalents. Moreover, it was possible for a well-equipped and experienced steel-fabricating shop to turn out welded structures at a sufficiently low cost per ton to turn the saving in weight into an overall saving in cost, and that despite the greater care needed in welded work, the high prices charged for good welding rods, and the cost of subjecting important seams to X-ray examination.

(2) In the case of road bridges decked with a reinforced-concrete roadway slab which was to be supported either on the upper flanges of the main girders or on those of intermediate longitudinal girders, or on both, the use of welding enabled such support to take the form of direct contact. With riveted girders that would be quite out of the question as it would prevent necessary provision being made for the replacement of loose rivets in service.

(3) There were many instances where frames, portals, and stanchions to carry heavy loads could not be constructed at all except by the use of welding.

(4) Apart from the technical and economic advantages of welding in bridge construction, it was to be preferred on account of the pleasing appearance obtainable, as compared with riveted girders. In the case of welded girders, butt joints could be used and the additional flange plates, where necessary, could be placed underneath the upper flange and above the lower flange, so that the outer surfaces were left perfectly straight.

(5) In skew bridges it was often desired, for architectural reasons, to arrange the cross-girders parallel to the abutments. By the use of welding the necessary skew connexions between the cross-girders and the main girders could be arranged in the simplest manner possible, whereas with riveted work their design left much to be desired, both technically and æsthetically.

Dr.-Ing. Schaper gave details of various bridges which had been built in Germany and illustrated his Paper with a large number of lantern-slides, some of which are reproduced with the Paper in *The Structural Engineer* for May, 1937.[1]

[1] Footnote 1, p. 203.

ANNUAL GENERAL MEETING.

11 May, 1937.

Sir ALEXANDER GIBB, G.B.E., C.B., F.R.S., President, in the Chair.

The Notice convening the Meeting was taken as read, as well as the Minutes of the Annual Meeting of the 12th May, 1936, which were confirmed and signed by the Chairman.

The following Report of the Council (pp. 216 *et seq.*) upon the Proceedings of The Institution during the Session 1936–37 was read, the Statement of Accounts (pp. 236 to 246) being taken as read.

The PRESIDENT moved—That the Report of the Council be received and approved, and that it be printed in the Journal of The Institution.

Mr. S. B. DONKIN seconded the motion, which was carried unanimously without any discussion.

The Scrutineers reported the election of the Council for 1937–38 as follows :—[1]

President.

SYDNEY BRYAN DONKIN.

Vice-Presidents.

William James Eames Binnie, M.A.
Sir Clement Daniel Maggs Hindley, K.C.I.E., M.A.
Maurice FitzGerald Wilson.
Sir Leopold Halliday Savile, K.C.B.

Other Members of Council.

Athol Lancelot Anderson, C.B.
David Anderson, LL.D., B.Sc.
Thomas Henry Bailey.
Asa Binns.
John Job Crew Bradfield, C.M.G., D.Sc., M.E. (*Australia*).
Raymond Carpmael, O.B.E.
Frederick Charles Cook, C.B., D.S.O., M.C.
Sir Harley Hugh Dalrymple-Hay.
Jonathan Roberts Davidson, C.M.G., M.Sc.
Thomas Peirson Frank.
Ralph Freeman.
Griffith John Griffiths.
William Thomson Halcrow.
Charles George Hawes, B.Sc. (*India*).

[1] The Council commence their term of office on the first Tuesday in November, 1937.

Roger Gaskell Hetherington, C.B., O.B.E., M.A.
Ralph Frederick Hindmarsh.
George McCausland Hoey, B.A., B.E. (*India*).
Professor Charles Edward Inglis, O.B.E., M.A., LL.D., F.R.S.
Alfred Dale Lewis, M.A. (*South Africa*).
William Henry Morgan, D.S.O.
Alexander Newlands, C.B.E.
Francis Ernest Wentworth-Sheilds, O.B.E.
Julian Cleveland Smith, LL.D. (*Canada*).
Reginald Edward Stradling, C.B., M.C., Ph.D., D.Sc.
Sir John Edward Thornycroft, K.B.E.
Hugh Vickerman, O.B.E., D.S.O., M.Sc. (*New Zealand*).

Dr. W. L. Lowe-Brown, proposing the resolution—That the thanks of the meeting be accorded to the Scrutineers of the ballot and that the ballot papers be destroyed—said the task of the scrutineers was a very difficult one because the Members took their duties in regard to voting so seriously; in fact he was told that some Members scratched out every name on the list and substituted other names of their own. He thought, however, there had been much less trouble in that respect in 1937 because there had been a large list of very suitable people, and the difficulty had been to know whom to choose. Mr. J. N. Reeson seconded the resolution, which was carried by acclamation.

Mr. J. D. C. Couper, in acknowledging the thanks accorded to the scrutineers, referred briefly to the increase in the number of candidates which amounted to 66 per cent. of the candidates who were not elected. As it was the habit of the scrutineers to count the votes which were not given and not those which were given, the increased labour was somewhat considerable. However, the scrutineers had survived it. He mentioned also that the system of counting votes which had been evolved by The Institution in the past, had been based upon the very sound assumption that scrutineers could not be expected to be able to add up a column of figures, and therefore the scrutineers were provided by The Institution with a mechanical counter which printed the totals. It had been customary in the past to rely implicitly on that machine, but unfortunately it had been found on this occasion to give incorrect figures, and that had also increased the work as the figures had to be checked by ordinary manual, or perhaps he ought to say capital, methods. All the columns had, however, been added up individually until two separate scrutineers had obtained the same result; and therefore he thought it could safely be said that the result of the ballot accorded with the votes cast.

Mr. H. J. F. Gourley proposed and Mr. Lloyd seconded the

resolution—That the thanks of The Institution be given to Messrs. P. D. Griffiths and E. W. Monkhouse, Auditors, and that they be re-appointed Auditors for the current financial year. The resolution was carried unanimously, and was acknowledged by Mr. Monkhouse.

Sir Cyril Kirkpatrick proposed "That the thanks of this meeting be accorded to Sir Alexander Gibb, President, for his conduct of the business as Chairman of the meeting." Mr. P. J. Cowan said that, as an old school-fellow of the President, he had great pleasure in seconding the motion. The motion was carried by acclamation.

The Chairman, in thanking the Members for the way in which they had received the motion, said that, although the present was his last appearance in the Chair as President, he could assure them that if there was anything he could do for the good of The Institution or for its Members in the future, he would certainly do it. He thanked the Members for their forbearance during his year of office, and congratulated them not only on electing Mr. Bryan Donkin as their President for next year but also on electing as their new Council the gentlemen they had chosen. It was an extremely good list. He desired to say about Mr. Bryan Donkin that that gentleman's grandfather had been one of Telford's most loyal supporters when The Institution had been started, and that the family had carried on in engineering ever since. It was therefore very fitting that they should have Mr. Bryan Donkin as their President for the ensuing year.

The proceedings then ended.

REPORT OF THE COUNCIL, 1936–37.

Coronation of King George.

In presenting the Annual Report upon the state of The Institution in accordance with the By-laws, the Council wish in the first place to take the opportunity afforded by the Coronation to record on behalf of the members of The Institution expressions of loyalty and congratulation to Their Majesties King George VI and Queen Elizabeth. Sixteen leading Engineering Institutions and Societies accepted the proposal made to them by the Council to submit a joint loyal Address of congratulation to Their Majesties.

Patronage.

His Majesty King George, who was, prior to his Accession, an Honorary Member of The Institution, has graciously consented to become its Patron, and the members will also be gratified to learn that His Royal Highness The Duke of Kent has accepted election into The Institution as an Honorary Member.

Ordinary Meetings.

The Opening Meeting was held on the 3rd November, when Sir Alexander Gibb delivered his Presidential Address. He took as his subject "Engineers and Empire Development," and dealt with the evolution of Engineering in its various branches, giving examples of outstanding works carried out at home and overseas which showed that Engineering had been the foundation of the British Empire—as it was bound to be of every empire. He concluded with a plea for closer co-operation between Engineering Institutions and the necessity for all to subordinate some of their more personal and independent views and feelings to a common policy.

Seventeen Ordinary Meetings have been held at which the following Papers were discussed (the discussion on two of these Papers each occupying an additional evening):—

Subject.	Author.
Road Design and Road Safety.	F. C. Cook, D.S.O., M.C., M. Inst. C.E.
Ship-Canals Utilized for Drainage.	L. R. Wentholt, D. Tech. Sc., M. Inst. C.E.
The Lower Zambezi Bridge.	F. W. A. Handman, C.B.E., M. Inst. C.E.
The Construction of the Lower Zambezi Bridge.	G. E. Howorth, M.C., B.Sc., M. Inst. C.E.
The Maintenance of Waterways to Harbours and Docks.	Raymond Carpmael, O.B.E., M. Inst. C.E.

Ordinary Meetings.

The Second-Stage Development of the Lochaber Water-Power Scheme.	A. H. Naylor, M.Sc., B.Sc. (Eng.), M. Inst. C.E.
Pre-Stressing Bridge Girders.	H. J. Nichols, B.Sc. (Eng.), M. Inst. C.E.
Salonika Plain Reclamation-Works.	B. W. Huntsman, B.Sc. (Eng.), M. Inst. C.E.
The Lake Copais, Boeotia, Greece: Its Drainage and Development.	A. J. Dean, B.Sc. (Eng.), Assoc. M. Inst. C.E.
Fundamental Research on the Application of Vibration to the Pre-Casting of Concrete.	D. A. Stewart.
Modern Developments in Broadcasting Transmission and Television.	Sir Noel Ashbridge, B.Sc. (Eng.), M. Inst. C.E.
West Middlesex Main Drainage.	D. M. Watson, B.Sc., M. Inst. C.E.
Welded Joints in Pressure-Vessels.	S. F. Dorey, D.Sc., M. Inst. C.E.
Kincardine-on-Forth Bridge.	J. Guthrie Brown, M. Inst. C.E.
[1]The Mechanics of the Voussoir Arch.	Professor A. J. Sutton Pippard, M.B.E., D.Sc., M. Inst. C.E., Eric Tranter, B.Sc., Stud. Inst. C.E., and (Miss) Letitia Chitty, M.A.
The Reconstruction of the Chester—Holyhead Road, near Penmaenmawr, North Wales.	C. L. Howard Humphreys, T.D., M. Inst. C. E.
The Flow of the River Severn, 1921–1936.	Professor S. M. Dixon, O.B.E., M.A., B.A.I., M. Inst. C.E., Gerald Fitzgibbon, B.A., B.A.I., and M. A. Hogan, D.Sc., Ph.D., M. Inst. C.E.

The awards for Papers read and discussed, for Papers published without oral discussion, and for Students' Papers will be announced in the October Journal. Awards.

Five Informal Meetings were held during the Session, at two of Informal Meetings.

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[1] "Films of Tests on Road Bridges," carried out for the Ministry of Transport, were also exhibited by Norman Davey, B.Sc. (Eng.), Ph.D., Assoc. M. Inst. C.E., of the Building Research Station.

Informal Meetings

which the attendances were much above the average. The subjects and names of the Introducers were as follows :—

Subject.	Introducers.
"Methods of Providing Permanent Non-Slip Surfaces for Roads."	H. S. Keep, M.C., B.Sc. (Eng.), Assoc. M. Inst. C.E.
"The Report of the Sea Action Committee."	John Shaw, B.Sc. (Eng.), Assoc. M. Inst. C.E.
"Conditions of Contract for Civil Engineering Work."	E. J. Rimmer, M. Eng., Barrister-at-Law, Assoc. M. Inst. C.E.
"Co-ordination of the Chemical and Engineering Aspect of Water-Supply, with special reference to Corrosion Problems."	G. Howard Humphreys, M.A., M. Inst. C.E.
"A Survey of the Programme of the Institution Research Committee."	A. H. Naylor, M.Sc., B.Sc. (Eng.), M. Inst. C.E.

Lectures.

Mr. J. L. Savage, Chief Designing Engineer, Bureau of Reclamation, United States Department of the Interior, delivered a Special Lecture on the 15th April, on "The Boulder Dam." The Lecture was illustrated by a complete set of motion pictures showing the progress during construction and the various methods of carrying out the work.

The Institution Lecture to Students on "Modern Methods of Structural Design" was delivered before the Association of London Students on the 9th December by Professor J. F. Baker, M.A., D.Sc., Assoc. M. Inst. C.E., and was repeated at Meetings of the Local Associations at Belfast, Birmingham, Bristol, Cardiff, Glasgow, Manchester, and Newcastle, the Lecture in some cases being read by a deputy, owing to the unavoidable absence of Professor Baker.

The Forty-Third James Forrest Lecture was delivered on the 27th April, by Sir William H. Bragg, O.M., K.B.E., D.Sc., P.R.S., Hon. M. Inst. C.E., on the subject of "The Crystal and the Engineer."

Joint Meetings.

Four Joint Meetings were held with other Institutions and engineering bodies as follows :—

28 October, with the British Section of the Société des Ingénieurs Civils de France, when a Paper on "A Study of the Underground Road Crossings of Paris" was read by Monsieur Gaston Bardet ;

11 February, with the British Section of the Société des Ingénieurs Civils de France and the Institution of Structural Engineers,

when a Paper on "The Construction of Large Modern Water Dams" by Monsieur A. Coyne, Ingénieur-en-Chef des Ponts et Chausées, was read and discussed; Joint Meetings.

2 March, with thirteen other Engineering bodies, in the Hall of the Royal Geographical Society, when a Symposium on Research in relation to the Motor Vehicle was read and discussed;

18 March, with the Institution of Structural Engineers, when Dr. Ing. Schaper, Director of the German State Railways, delivered a Lecture on "The New German Railway Bridges."

Students' Meetings and Visits.

The Association of London Students opened its Session on the 18th November, when Mr. Geoffrey Wood, B.Sc. (Eng.), Stud. Inst. C.E., Chairman of the Association, gave an Address outlining the history of constructional design. Sir Alexander Gibb, President Inst. C.E., was in the Chair. In addition to the Institution Lecture, Papers were read at five Meetings, an Informal Meeting was held, and a lantern lecture on the Work of the Metropolitan Water Board was given by Mr. H. F. Cronin, M.C., B.Sc. (Eng.), M. Inst. C.E. The Annual Dinner of the Association was held at the Connaught Rooms on the 19th February, when the President was the Guest of the evening; 73 members of the Association and their guests attended. Visits were paid to six places of engineering interest and it is satisfactory to record that the attendances were good. 582 Students are members of the Association.

Local Associations.

The Glasgow Association of Students has held seven meetings, including one at Edinburgh, and the Council note with satisfaction that of the five Papers read two were by Students of the Association. Four visits have taken place, and the Annual Dinner of the Association was held on the 2nd February, the company numbering 150. Students of The Institution affiliated to the Association number 184, this being an increase of 10 upon last year.

A meeting of the Students of The Institution was held in Manchester under the auspices of the Manchester and District Association from 24–26 September, 1936. In addition to members of the Association, members and Students with their ladies from all parts of the country took part in the meeting. The programme consisted of meetings at which two Papers by Students were read and discussed, a number of visits to engineering works and places of interest, and social gatherings. This new departure of a summer meeting held at a provincial centre proved a great success, and it is hoped that meetings of a similar character will be held at other provincial centres in the future.

The roll of the Manchester and District Association numbers 133 Corporate Members and 128 Students, the latter figure being an

Local Associations.

increase of 20 over last year. Eleven meetings have been held, at one of which a Paper was read by a Student, and members of the Association have also attended by invitation meetings of the North Western Branch of the Institution of Mechanical Engineers and the North Western Area Branch of the Illuminating Engineering Society. The Annual Dinner was held on the 3rd February, when the attendance was 107.

The roll of the Birmingham and District Association numbers 191 Corporate Members and 152 Students, and again shows a considerable increase over last year in the number of Students. Eleven meetings have been held, including two Students' meetings and a joint meeting with the local branches of the Institutions of Mechanical and Electrical Engineers, and one Paper by a Student has been read. Five visits to works have taken place. The Annual Dinner was held on the 10th December, 1936, when 115 members and guests attended.

The Newcastle-upon-Tyne and District Association numbers 120 Corporate Members and 66 Students, as compared with 131 Corporate Members and 57 Students last year. Eight meetings have been held at Newcastle and six at Stockton, but the Council regret to note that no Papers by Students have been read before the Association this Session. The Annual Dinner was held on the 14th January and the attendance was 99, including 25 Student Members of the Association.

The Yorkshire Association shows an increase of 11 in its membership, and now numbers 208 Corporate Members and 85 Students. Eight meetings have been held during the Session, and at one of these the Author of the Paper was a Student. One visit to works has been paid, and a number of visits have been arranged for the Summer months. The Annual Dinner was held on the 15th January at Sheffield and was attended by 70 members and guests.

The Bristol and District Association has 73 Corporate Members and 85 Students on its roll, compared with 80 Corporate Members and 72 Students last Session. Six meetings have been held, including one again at Gloucester, with very satisfactory results. The Annual Dinner was held on the 27th January, when 103 members and their guests attended.

The South Wales and Monmouthshire Association, which numbers on its roll 150 Corporate Members and 57 Students, has held seven meetings during the Session at Cardiff and one at Swansea, and in September last some 30 members took part in a visit to the reservoirs of the Cardiff Corporation and the waterworks under construction for Newport Corporation at Talybont, Breconshire. The first Annual Dinner of the Association was held at Cardiff on the 2nd March, when 60 members of the Association and guests were present.

Local Associations.

The Northern Ireland Association shows an increase in its membership, and now numbers 84 Corporate Members and 23 Students, compared with 83 Corporate Members and 17 Students last Session. Eleven meetings have been held, including one meeting at which a Paper by a Student was read, and two visits to works were made during the Summer of 1936. The Annual Dinner was held on the 11th December, when 81 members and guests attended.

All the Local Association Dinners at home were attended by Sir Alexander Gibb as President, and by Mr. E. Graham Clark, representing the Secretary.

The Council are pleased to note an increase of 27 per cent. in the membership of the Buenos Aires Association, which now numbers 92 Corporate Members and 17 Students. Four meetings have been held during the year, at one of which four brief Papers were submitted by four Students, and a number of visits to works of engineering interest have taken place. The Association has accepted a sum of money from Sir Follett Holt, K.B.E., M. Inst. C.E., the income from which is to be devoted to the provision of an award for the best contribution of the year relating to the design or maintenance of track. The Annual Dinner of the Centre of British Engineering and Transport Institutions in Buenos Aires was held on the 21st August, 1936, when the members were honoured by the attendance of His Majesty's Ambassador, Sir Neville Meyrick Henderson, and Dr. Manuel Castello, President of the Argentine Centre of Engineers.

The membership of the Malayan Association in October, 1936, was 109 Corporate Members and 3 Students, as compared with a total of 106 for the previous year. The Association had the pleasure of welcoming and entertaining Mr. W. J. E. Binnie, Vice-President, to an informal dinner during a visit he paid to Singapore in January, 1937. During the Session, six meetings were held and six visits were paid to engineering works. As in previous years, these meetings and visits were held in conjunction with the Engineering Association of Malaya.

The Shanghai Association, in co-operation with the local centres of the Institutions of Mechanical and Electrical Engineers and the Engineering Society of China, has held sixteen joint meetings and eight joint visits to works of engineering interest. Two Papers were read by members of The Institution. In addition, a meeting of the Association was held at which the Chairman (Mr. C. D. Pearson) presented his Address. The Annual Dinner and Annual General Meeting will be held in May. There are 27 Members and 1 Student on the roll of the Association.

Research.

The work of the Research Committee has continued to grow during the year and valuable co-operation has been secured with other

Research.

Engineering Institutions and bodies in research work of common interest. Investigation work in connexion with the following subjects has been continued :—

Wave-pressures on sea structures,
Vibrated concrete (jointly with the Institution of Structural Engineers),
Pile-driving (jointly with the Federation of Civil Engineering Contractors),
Earth-pressures,
Special cements (jointly with the British Committee on Large Dams of the World Power Conference),
Fish passes,
Velocity formulas for open channels and pipes,

and the scope of work on *Corrosion in soils* has been widened by separating the work on corrosion of metals from that of corrosion of cement products. In regard to the former, co-operation has been obtained with the Corrosion Committee of the Iron and Steel Institute and the Non-Ferrous Metals Research Association, and in regard to the latter, co-operation and financial assistance have been obtained with various bodies interested in cement. Good progress has been made in the preparation of draft regulations in respect of

Breathing apparatus for use in sewers,
Earthing to metal water-pipes and mains,

and in the drawing up of a code of practice for

Reinforced-concrete structures for the storage of liquids.

Work has commenced on a research into

Repeated stresses in structural elements,

and on the drawing up of a code of practice for

Simply-supported steel bridges (jointly with the Institution of Structural Engineers), and in this connexion practical investigation is being undertaken into wind-pressures on bridges;

while the drawing up of a code for

Steel structures (jointly with the Institution of Structural Engineers) is about to commence.

The first Interim Report of the Joint Sub-Committee on Vibrated Concrete has appeared during the year. Articles dealing with the research on Earth Pressures which have been published in the Journal include a note on the shearing resistance of soils and a report

by Mr. L. F. Cooling on the First International Conference on Soil Mechanics and Foundation Engineering held at Harvard University. Research.

In connexion with the research on Special Cements a note on special cements for mass-concrete structures and their specification has been prepared by Mr. W. T. Halcrow and Dr. F. M. Lea, the latter of whom attended the Second International Congress on Large Dams of the World Power Conference at Washington.

The programme of the Research Committee formed the subject of discussion at an Informal Meeting of members of The Institution on 17th March, when valuable criticisms and comments on the work were contributed by those present.

The Research Committee has continued during the year to examine draft British Standard Specifications which have been referred to The Institution for comment.

Sea-Action Committee.

The Committee on the Deterioration of Structures exposed to Sea-Action have published their 16th (Interim) Report describing the progress of the investigations during the past year.

Reports have been received on periodical inspections of timber specimens impregnated with various substances and exposed in different parts of the world, and specimens have been sent home for examination by Professor George Barger, F.R.S., from Auckland (N.Z.), Mauritius, Singapore and Takoradi. Incised and creosoted specimens have been received from Colombo after an exposure of 2½ years. In addition, specimens of various untreated and creosoted timbers exposed at Leith have been received.

Reports on periodical inspections of iron and steel specimens at Auckland, Plymouth, Halifax and Colombo have been received. These specimens are now being returned after 15 years' exposure, and Auckland and Halifax specimens have already been examined and reported upon by Dr. J. Newton Friend.

Periodical reports on reinforced-concrete specimens exposed at Sheerness and Watford have been received from the Building Research Station.

Factories Bill.

The Council having learned last autumn that a new Factories Bill was to be introduced in Parliament and that it was proposed that works of engineering construction be brought within its scope, a Committee was set up with power to co-operate with the Federation of Civil Engineering Contractors for the purpose of making any representations to the Home Office which appeared desirable on the subject of the proposed Bill.

It was decided not to press for the total exclusion of works of engineering construction from the provisions of the Bill and a letter was addressed to the Home Secretary stating that The Institution wished to support the legislation proposed, but asking that certain

Factories Bill.

questions of principle be favourably considered when the Bill was being drafted. The most important of these principles was that the design and methods of execution of works of engineering construction should be under the sole control of the engineer in charge, and that nothing in the Bill should interfere with such control.

The Home Office has given sympathetic consideration to the representations made to them, and, in drafting the Bill, has met the points raised. The Council have taken steps to have the Bill watched during its passage through Parliament.

Architects' Registration.

The Council have also had under consideration proposed legislation to restrict the use of the name "Architect" to registered Architects and have informed the Architects Registration Council of the United Kingdom, which has promoted the Bill in Parliament, that it is not proposed to oppose the enactment of this Bill, but that The Institution would strongly oppose any powers which might be sought to preclude engineers from designing, planning or constructing buildings, structures, etc.

Arbitration.

The Arbitration Bill at present before Parliament has received the Council's attention, and they are in general agreement with the objects which the Bill seeks to secure.

Legislation affecting The Institution.

It may be mentioned that, in respect of the promotion of legislation in Parliament, the Council have given authority to the President to make arrangements through the Institution solicitor for the employment of Parliamentary Agents should the need arise in connexion with any Bill before Parliament which may affect the interests of The Institution.

Proposed L.C.C. Building By-Laws.

The Building By-laws and By-laws for the Use of Timber in the Construction and Conversion of Buildings proposed to be made by the London County Council under the London Building Act (Amendment Act), 1935, have been examined by a joint committee of representatives of The Institution, the Royal Institute of British Architects, the Chartered Surveyors' Institution and the Institution of Structural Engineers. Conferences on the subject followed with representatives of the London County Council, and amended copies of the By-laws have since been submitted. It is hoped that By-laws will shortly be settled which will be satisfactory both to the professional Institutions concerned and to the London County Council.

Engineering and the Public.

The Council have had under consideration for some time the question whether steps should be taken in the interests of Engineering generally to place before the general public information concerning the science and practice of Engineering and its services to the public, as their experience and the inquiries they had made showed that there existed a lack of understanding of the value of Engineering Science. It was accordingly decided to consult other Engineering

Engineering and the Public.

Institutions and bodies, and, as the result of a joint meeting held at this Institution in December, 1936, it was decided to form an Engineering Public Relations Committee, of which the first meeting was held on the 27th April, 1937.

Engineering Congress, Glasgow, 1938.

The Council have accepted an invitation received from the Institution of Engineers and Shipbuilders of Scotland to co-operate with the principal Engineering Societies in Great Britain in organizing an International Engineering Congress at Glasgow, to be held in that city during the Empire Exhibition, Scotland, 1938. A general committee consisting of representatives of these Societies has been appointed and will be responsible for all arrangements in connexion with the Congress.

The Institution has undertaken to provide accommodation and secretarial services for a London Committee, which, with a Committee in Glasgow, will act in an executive capacity to the General Committee.

"Engineering Abstracts."

Beginning with the June Number, 1936, "Engineering Abstracts" appeared at two-monthly intervals instead of quarterly. The future of "Engineering Abstracts" is now under consideration in conjunction with other Engineering Institutions and with the Department of Scientific and Industrial Research, with a view to increasing their value to the engineer and scientific worker.

Co-operation.

The extent to which the Council is seeking the co-operation of engineering institutions and of other societies and associations will be noted in this record of The Institution's activities. The Council will continue to work for closer co-operation by holding joint meetings, by promoting conjointly engineering research and by any other means which may lead to the greater co-ordination of effort in the advancement of the objects for which the Engineering Institutions were constituted.

Nominations and Appointments.

Various nominations and appointments have been made or renewed by the Council during the past year, and The Institution is or has been represented on advisory or administrative bodies and committees by the following members of The Institution:—

Royal Commission for Exhibition of 1851	The President.
Grant Committee of the Royal Society	The President.
General Board of National Physical Laboratory	Sir Alexander Gibb, G.B.E., C.B., F.R.S. Sir Richard A. S. Redmayne, K.C.B.
Admiralty Selection Board for the Appointment of Assistant Civil Engineers	Sir Alexander Gibb, G.B.E., C.B., F.R.S. Sir Cyril R. S. Kirkpatrick.

Nominations and Appointments.

Committees of the Department of Scientific and Industrial Research :—

Committee on Testing Work for the Building Industry	The President or his Deputy.
Mechanisation Board, Army Council	W. G. Wilson, C.M.G.
Advisory Panel on Transport (Ministry of Transport)	O. R. H. Bury. Col. R. E. B. Crompton, C.B., F.R.S. Sir John P. Griffith. J. A. Saner.
Ministry of Health Technical Committee on Materials for the Economic Construction of Flats	W. L. Scott.
Home Office Sub-Committee on Air Raid Precautions	Sir Leopold H. Savile, K.C.B.
Science Museum Advisory Council, Board of Education	Professor C. E. Inglis, O.B.E., F.R.S.
Court of the University of Bristol	Raymond Carpmael, O.B.E.
Court of the University of Liverpool	Thomas Molyneux, O.B.E.
Court of the University of Sheffield	Sir William H. Ellis, G.B.E.
Governing Body of the Imperial College of Science and Technology	Sir George W. Humphreys, K.B.E.
Council of the City and Guilds of London Institute	The President.
Court of University College, Southampton	F. E. Wentworth-Sheilds, O.B.E. A. C. Hughes.
Governing Body of the School of Metalliferous Mining, Cornwall	J. G. Lawn, C.B.E.
Thomason College, Roorkee, Advisory Council	G. McC. Hoey.
Old Centralians Committee on Memorial to Dr. W. C. Unwin	Sir Richard A. S. Redmayne. Sir Charles L. Morgan, C.B.E. The Secretary.
Engineering Advisory Committee, Huddersfield Engineering College	V. Turner. J. Urquhart.
Technical Advisory Committee of the Institute for Research in Agricultural Engineering at the University of Oxford	Sir John E. Thornycroft, K.B.E.

Nominations and Appointments.

Engineering Joint Council	Sir George W. Humphreys. Sir Clement D. M. Hindley, K.C.I.E.
Parliamentary Science Committee	Sir George W. Humphreys. Sir Murdoch MacDonald, K.C.M.G., C.B., M.P.
Council of the London Society	T. H. Bailey.
Governing Body of the Denning Trust	A. E. Cornewall-Walker.
Tribunal of Appeal, London Building Act, 1930	Sir Cyril R. S. Kirkpatrick.
Royal Institute of British Architects Sub-Committee on Information Bureau on Building Materials and their Uses	R. E. Stradling, C.B., M.C., Ph.D., D.Sc.
Joint Committee on Materials and their Testing	R. H. H. Stanger.
Alloys and Iron Research Committee of the Institution of Mechanical Engineers	Sir Robert A. Hadfield, Bt., F.R.S.
British Cast Iron Research Association	Sir Robert A. Hadfield.
Permanent Commission of International Navigation Congresses	Sir Cyril R. S. Kirkpatrick. N. G. Gedye, O.B.E.
Permanent International Association of Road Congresses, British Organizing Committee	F. C. Cook, C.B., D.S.O., M.C.
Ministry of Health Advisory Committee on revision of existing model building by-laws, under Public Health Act, 1936	B. L. Hurst.
Institution of Mechanical Engineers, General Committee, Lubrication and Lubricants	S. B. Donkin.
World Power Conference British National Committee	S. B. Donkin.
World Power Conference International Sub-Committee on Special Cements	W. T. Halcrow.
International Electrotechnical Commission on Steam Turbines	I. V. Robinson.
International Electrotechnical Commission Advisory Committee on Internal-Combustion Engines	W. A. Tookey.
International Association for Testing Materials Organizing and Reception Committee	R. H. H. Stanger.

Nominations and Appointments.

The Institution is represented as follows on Councils of the British Standards Institution :—

General Council	Sir Cyril R. S. Kirkpatrick.
Engineering Divisional Council	S. B. Donkin. R. G. Hetherington, C.B., O.B.E. A. Newlands, C.B.E.

and has also representatives on numerous Committees, Sub-Committees and Panels.

Main Committee of the Canadian Engineering Standards Association	H. H. Vaughan.

The Council have appointed Sir Alexander Gibb, G.B.E., C.B., F.R.S., President Inst. C.E., to represent The Institution at the Celebration of the 50th Anniversary of the foundation of the Engineering Institute of Canada, to be held in Montreal, the 15th to the 18th June, 1937. In view of the fact that the President, Sir Alexander Gibb, would be attending the Coronation Service in another capacity, the Council appointed Mr. S. B. Donkin, Senior Vice-President, to represent The Institution at Westminster Abbey.

Ewing Medal.

The Council have instituted a gold medal, to be known as the James Alfred Ewing Medal, in memory of the late Sir Alfred Ewing, K.C.B., F.R.S. (1855–1935), Honorary Member, to be awarded by the Council to a person whether a member of The Institution or not, for specially meritorious contributions to the science of engineering in the field of research. An endowment fund, contributed jointly by Lady Ewing, by The Institution, and by friends and admirers of the late Sir Alfred Ewing, has been created, and will be administered by the Council.

Dugald Clerk Lecture.

The Council have decided to perpetuate the memory of the late Sir Dugald Clerk, K.B.E., F.R.S. (1854–1932), Past Vice-President, by naming after him the biennial lecture (hitherto known as the "Institution Lecture to Students") delivered to Students of The Institution in London and in the provinces. The "Dugald Clerk" Lecture will in future alternate annually with the "Vernon Harcourt" Lecture.

Scholarships.

The William Lindley Scholarships and the Palmer Scholarship remain vacant and the Council hope that during the forthcoming year eligible candidates may be found to fill these vacancies. A C. C. Lindsay Civil Engineering Scholarship of £25 per annum for 4 years has been awarded to Mr. Gavin Harvie, Stud. Inst. C.E.

Charles Hawksley Prize.

The Council having received the report of the judges of the Charles Hawksley Prize Competition for 1937, have awarded a Charles

Hawksley Prize of £150 to Mr. Frank Robert Bullen, B.Sc. (Engineering) (London), Assoc. M. Inst. C.E., of Beckenham, for his design of a tidal wharf. Two other competitors have been honourably mentioned and will receive sums of £50 each in recognition of their good work. The subjects set were a fly-over road junction, a railway station with a car park, and a tidal wharf. Eleven entries were received, this number being below the average for recent years. Charles Hawkesley Prize.

Reference was made in last year's report to the demolition of the premises known as No. 1 Great George Street, and to the completion of the N.W. corner of the Institution building. It is expected that the work will be finished within the course of the next few weeks. Building.

Advantage has been taken of the rebuilding to place on permanent exhibition at the west end of the main library the collection of books and other documents presented to The Institution by Thomas Telford with the object of forming the foundation of an engineering library, when he accepted the invitation to become the first President. This collection is augmented by a number of other books and two oil paintings which he bequeathed to The Institution. In this way it is intended to acknowledge the great debt which The Institution owes to Telford in supplying the nucleus of a library which was to grow into the valuable collection now in the possession of The Institution.

In order to perpetuate the memory of Dr. W. C. Unwin, F.R.S. (1838–1933), the Council have ordered that his name be carved in the stonework on the main landing.

The Accounts for the year ending 31st March, 1937, which have been duly audited, are detailed in Appendix II of this Report, and may be summarised briefly as follows— Accounts.

The *Total Income* for the year amounting to . . (as compared with £43,915 last year) included £100 Life Composition and £203 for Income Tax recovered. Subscriptions, Entrance and Examination fees totalled £41,719 (compared with £41,320 last year) and Dividends and Interest received amounted to £1,424 as compared with £2,134 last year.	£43,411
The *Total Expenditure* charged against the year's Income amounted to . . . . (as compared with £42,618 last year) but it included Provisions of £12,600 (viz. £10,000 for Publications Account and £2,600 for Research Reserve) as compared with £11,000 last year.	£44,095
The *Revenue Account* therefore results in an *adverse balance* of	£684

Accounts.

which has been carried to the debit of the General & Contingency Reserve, reducing the credit balance thereon from £1,297 to £613 as shewn in the Balance Sheet.

The actual expenditure incurred during the year on "Publications Account" amounted to £18,159 (compared with £14,625 last year), but was relieved by credits for Advertisements, Sales, etc., of £4,494 (against £2,210 last year), leaving the net expenditure for the year at £13,665 (compared with £12,415). This amount, however, was £3,665 more than the £10,000 Provision credited, thereby increasing the overspent balance on this Account (from £2,415 last year) to £6,080 at 31st March, 1937, as shown by the Balance Sheet. This excess expenditure falls to be liquidated in the future.

The *Research Reserve* credit balance has been augmented by £1,007 during the year, viz. from £1,200 to £2,207.

The expenditure incurred amounted to £1,882, whereas the credits (made up of the appropriation from Revenue Account of £2,600, and contributions by Outside Bodies of £289) totalled £2,889.

On Trust Funds Income Account there was received a total of £1,235 (£1,224) and the Expenditure amounted to £735 (£977).

Contributions amounting to £656 (£630) were received from Home and Overseas Harbour and Dock Authorities towards the cost of the research into the Deterioration of Structures exposed to Sea Action. The expenditure during the year was £840 (£486).

Library.

During the year 737 volumes were presented to the Library and 180 were purchased, making a total, on the 31st March, 1937, of 62,300.

The number of applications received for books on loan continues to increase, the figure for the year being 1,983, an increase over 1935–36 of 251. The demand for the Loan Library catalogue continues.

Tait Room.

The Council have decided to use the sum bequeathed to The Institution by the late Mr. W. A. P. Tait (1866–1929), Vice-President, in furnishing a room, formerly known as the "Sea Action Museum," as a library containing books on legal matters affecting engineers, and as a permanent record of the interest which Mr. Tait always displayed in the Institution library. A room on the lower ground floor has been converted into a museum for the permanent exhibition of the collection of specimens illustrating the deterioration of structures exposed to sea action.

Gifts.

Mr. John D. Watson has presented a portrait of himself, painted by Mr. Clive Gardiner, and Lady Palmer has presented two paintings by Wyllie, one of the London Dock and the other of King George V

Dock—works with which the late Sir Frederick Palmer was intimately connected. Gifts.

Other gifts received by The Institution during the year include a silver model of a Chinese bridge, from Colonel C. H. Wilmer; a silver casket containing an Address presented to the late Mr. F. Gascoigne Lynde on his retirement after 15 years of railway service in India, from Mrs. E. E. Lynde; a mezzotint engraving by Macbeth-Raeburn, R.A., of the portrait of Thomas Telford by Raeburn, from Sir Alexander Gibb; and silver impressions from the dies of the Telford Medal, formerly the property of Joshua Field, Past-President, presented by Miss M. G. Field.

Smeatonian Society.

The Council have acceded to a request of the Smeatonian Society of Civil Engineers to afford permanent accommodation in the Institution building for books, pictures and other relics, the property of that Society.

Use of Building.

The Council have been pleased to afford accommodation in the Institution premises for meetings for various purposes to the following bodies:—

The Institution of Gas Engineers.
The Institution of Chemical Engineers.
The Institution of Structural Engineers.
The Institution of Water Engineers.
The Iron and Steel Institute.
The Society of Chemical Industry.
The Permanent Way Institution.
King's College Engineering Society.
The Association of Consulting Engineers.
The Institute of Welding.
The Smeatonian Society.
The Newcomen Society.
The Board of Studies of the University of London.
International Congress on the Testing of Materials.
The British Standards Institution.

The Institution of Mechanical Engineers, The Chartered Surveyors' Institution and the Civil Service Commission have had the use of certain rooms for examination purposes, while the Ministry of Transport and the Board of Trade have held various official enquiries at the Institution.

Annual Dinner.

The Annual Dinner, at which The Institution was honoured by the presence of H.R.H. The Duke of Kent, K.G., Honorary Member, and of the Rt. Hon. the Lord Mayor of London and the Sheriffs, was, by kind permission of the Rt. Hon. the Lord Mayor and the Corporation of the City of London, held at Guildhall on Wednesday, 24 March, and was attended by 726 members and guests.

Conversazione.

A Conversazione was held at the Institution on the 10th June, 1936, and was attended by 2,640 members and guests.

Elections, Transfers, Admissions.

During the Session which ended on the 30th April, 1937, 498 proposals for election (including cases postponed from previous years) and 88 recommendations for the transfer of Associate Members to the class of Members have been considered by the Council.

For the year which ended the 31st March, 1937, the elections comprised five Honorary Members, namely, H.R.H. The Duke of Kent, K.G., K.T., G.C.M.G., G.C.V.O., Field-Marshal Sir William Riddell Birdwood, Bt., G.C.B., G.C.S.I., G.C.M.G., C.I.E., D.S.O., M.A., LL.D., D.C.L., Sir William Henry Bragg, O.M., K.B.E., M.A., D.Sc., Sc.D., LL.D., D.C.L., P.R.S., Dr. Charles Prosper Eugene Schneider, and Sir Robert Abbott Hadfield, Bt., D.Sc., D.Met., F.R.S.; 17 Members, 297 Associate Members, and 1 Associate; 454 candidates were admitted as Students, and the names of 15 Associate Members and 2 Students were restored to the Roll. From this addition of 791 must be deducted the deaths, resignations and erasures during the year; the member elected an Honorary Member and the Students elected Associate Members, amounting to 397 in all, showing a net increase of 394; 90 Associate Members have been transferred to the class of full Members.

Examinations.

The number of candidates presenting themselves for the October, 1936, Examination was 434, namely, 97 for the Preliminary Examination and 337 for the Associate Membership Examination. The entries for the April, 1937, Examinations were 704—164 for the Preliminary Examination and 540 for the Associate Membership Examination.

Bayliss Prizes of the value of £15 have been awarded to Mr. Kenneth Frederick Geesin, Stud. Inst. C.E., and to Mr. John Macdonald Gordon Forsyth, in respect of Sections A and B of the Associate Membership Examination for April and October, 1936, respectively, and Sheikh Basheer Ahmed has received honourable mention in connexion with the former examination.

The Roll.

The Roll of The Institution on the 31st March, 1937, stood at 11,748, the changes which took place in it during the year ended on that date being shown in the Table on the opposite page :—

The Roll at this date is 11,784.

The Council record with especial regret the deaths of *H.R.H.* Purachatra, Prince of Kambaeng Bejra, Siam, Honorary Member; *Sir* John Audley Frederick Aspinall, D.Eng., and *Sir* Brodie Haldane Henderson, K.C.M.G., C.B., Past-Presidents; *Professor* William Ernest Dalby, M.A., B.Sc., F.R.S., Vice-President; *Sir* Archibald Denny, Bart., LL.D., former Vice-President, and Kenneth Alfred Wolfe Barry, O.B.E., Member of Council.

The Roll.

—	1 April, 1935, to 31 March, 1936. Honorary Members.	Members.	Associate Members.	Associates.	Students.	Totals.	1 April, 1936, to 31 March, 1937. Honorary Members.	Members.	Associate Members.	Associates.	Students.	Totals.
Numbers at commencement	16	2222	6937	56	1676	10,907	14	2261	7121	58	1900	11,354
Transfers—Associate Members to Members	..	+98	−98	..	..		..	+90	−90	..	..	
Member to Associate Member		..	..	..	..		..	−1	+1	..	..	
Elections	1	11	399	4	..	+819	5	17	297	1	..	+791
Admissions	..	..	..	..	387		..	..	..	..	454	
Restored to Roll	..	1	15	..	1		..	..	15	..	2	
Deceased	2	53	57	2	4		1	60	67	1	2	
Resigned	..	13	47	..	19		..	14	44	1	20	
Erased	1	5	27	..	11		1	4	32	..	14	
Elected as an Honorary Member	..	..	..	..	..		..	1	..	..	..	
Elected as Associate Members	..	..	..	..	122	−372	..	..	..	..	129	−397
Removed—over age	..	..	..	..	3		..	..	..	..	..	
Failed to complete	..	..	1	..	1		..	.	2	..	3	
Failed to comply (Studentship)	..	..	..	..	4	+447	..	..	..	..	1	+394
Numbers at termination	14	2261	7121	58	1900	11,354	17	2288	7199	57	2187	11,748

Deaths.

The full list of deaths is as follows (*E.* refers to election, *T.* to transfer, and *A.* to admission) :—

Honorary Member (1).—*H.R.H.* Purachatra, Prince of Kambaeng Bejra, Siam (E. 1920).

Members (60).—Alfred Edmond Adie (E. 1896. T. 1902); Herbert Edward Allen (E. 1893. T. 1900); Jonathan Angus (E. 1887. T. 1894); *Sir* John Audley Frederick Aspinall, D.Eng. (E. 1881. T. 1887) (*Past-President*); Kenneth Alfred Wolfe Barry, O.B.E. (E. 1905. T. 1913) (*Member of Council*); James Herbert Bartlett (E. 1882. T. 1893); Onward Bates (E. 1888. T. 1895); Thomas Bennett (E. 1897. T. 1907); William Walls Bishop (E. 1932); Alfred Thomas Blackall (E. 1911); Reginald Brown, M.B.E. (E. 1897. T. 1913); John Hugh Buchanan (E. 1899. T. 1924); Henry Robert John Burstall, M.B.E. (E. 1888. T. 1897); Harold Edward Byrne, B.A.I. (E. 1907. T. 1914); Robert Edden Commans (E. 1883. T. 1895); John Peachey Crouch (E. 1899. T. 1910); *Professor* William Ernest Dalby, M.A., B.Sc., F.R.S. (E. 1894.

Deaths.

r. 1898) (*Vice-President*); Martin Deacon, O.B.E. (E. 1906. r. 1913); *Sir* Archibald Denny, *Bart.*, LL.D. (E. 1900) (*former Vice-President*); Peter Dodd, M.B.E. (E. 1890. r. 1910); Alexander John Dudgeon (E. 1904); *Professor* George Forbes, M.A., F.R.S. (E. 1883. r. 1889); Reginald Gadsby (E. 1914. r. 1936); Ernest Alfred Glanville, M.B.E. (E. 1906. r. 1930); John William Griffith, M.A., M.A.I. (E. 1902. r. 1915); Samuel Hall (E. 1914. r. 1936); Samuel Hare (E. 1917); *Sir* Brodie Haldane Henderson, K.C.M.G., C.B. (E. 1894. r. 1899) (*Past-President*); Henry Ashton Hill (E. 1887. r. 1896); Richard Henry Horsfield (E. 1928); *The Rt. Hon. Lord* Invernairn of Strathnairn (E. 1919); Claude William Kinder, C.M.G. (E. 1878. r. 1889); Walter Burditt Leane, M.C. (E. 1905. r. 1913); Stephen Leggett (E. 1904. r. 1916); Thomas Brewer Mather (E. 1922); Frank Merricks, C.B.E. (E. 1898. r. 1922); *Sir* Philip Arthur Manley Nash, K.C.M.G., C.B. (E. 1918); William Henry Owen (E. 1913); Francis Tucker Patterson (E. 1909. r. 1921); George Stephens Perry (E. 1900. r. 1912); John Kyle Prendergast, M.E. (E. 1920. r. 1936); Nicholas Kingswell Prettejohn, D.S.O. (E. 1928); Harry Alfred Richardson (E. 1897. r. 1917); Martyn Noel Ridley (E. 1886. r. 1912); Henry Fillmer Rutter (E. 1886. r. 1901); Arthur Molyneux Sillar, M.B.E. (E. 1906); David Carnejy Simpson (E. 1882. r. 1893); *Sir* Thomas Sims, C.B. (E. 1904); Alphonse Steiger (E. 1900); Hugh Stowell (E. 1906. r. 1913); Charles Lewis Strobel (E. 1889); Harold Blake Taylor, C.B.E., F.C.H. (E. 1887. r. 1896); William James Taylor, O.B.E. (E. 1889. r. 1902); *Professor* Elihu Thomson (E. 1895); Robert Henry Thorpe (E. 1886. r. 1911); *Professor* Vsevolod de Timonoff (E. 1890. r. 1916); Walter Clifford Tyndale (E. 1890. r. 1896); Henry Wardale (E. 1911); Horace Wilmer (E. 1877. r. 1888); Maurice Wilson (E. 1888. r. 1925).

Associate Members (67).—Alfred Ernest Abbott (E. 1907); Affonso de Oliveira de Albuquerque-Maranhão (E. 1892); Luis Andreoni (E. 1891); Henry Dudley Arnott (E. 1902); Francis Henry Ashhurst (E. 1873); Hedley Merriman Askew (E. 1936); Edward Henry Beckett (E. 1893); Arthur Begleman (E. 1933); Andrew Walker Bell (E. 1897); William Murray Binny (E. 1896); Percy St. John Bishop (E. 1908); Harry Hamilton Bowack (E. 1909); Maurice Francis Brick, M.A., B.A.I. (E. 1936); John Brown, B.Sc. (E. 1936); Thomas Patrick Browne, B.Sc. (E. 1936); Frank Buckley, B.Sc. (E. 1932); *Sir* Henry Parsall Burt, K.C.I.E. (E. 1883); James Butler (E. 1884); Vernon Cooper (E. 1902); Sherard Osborn Cowper-Coles (E. 1896); Robert Laird Mackie Dick, B.Sc. (E. 1927); Frederick Richard Dixon (E. 1918); Alexander Walker Duncanson, B.Sc., B.Eng. (E. 1907); Robert Cyril Fernando (E. 1914); George Swainston Findlay (E. 1925); Andrew Forbes (E. 1888); George James Furness (E. 1893); Richard Douglas Gauld, M.Eng. (E. 1922); David Ginsberg, B.Sc. (E. 1921); Charles Currie Gregory (E. 1876); Eric Chester Hillman, M.C., B.Sc. (E. 1924); John Lawrence Hodgson, B.Sc. (E. 1910); Arthur Honeysett (E. 1884); Richard Hosken (E. 1897); John Hunter, O.B.E. (E. 1879); Arthur Peregrine Pepperel Hutton, M.A. (E. 1930); William Arthur Ingham (E. 1926); Walter Roberts Jones (E. 1876); Arthur Francis St. John Kinsey (E. 1919); Sidney James Kirby (E. 1914); John Lee, M.A. (E. 1890); Alfredo Lisboa (E. 1886); Bell George Lloyd (E. 1891); John Long (E. 1927); William Ramsay McNab, Jun., B.Sc. (E. 1931); Arthur Henry Morgan (E. 1892); William Richard Wilmot Morgan (E. 1899); Robert Hutchison Murray (E. 1912); Cormac Seaghan O'Connell, B.E. (E. 1934); John Trevor Owen, M.A. (E. 1932); *Sir* Joseph Ernest Petavel, K.B.E., D.Sc., F.R.S. (E. 1901); Charles Albert Pollock (E. 1913); Henry Victor Prigg (E. 1891); William Henry Raven (E. 1913); Reenan Jacob van Reenan, B.A. (E. 1916); Leonard Henry Richards (E. 1919); Charles Edwin Rivers (E. 1900); Narain Singh Sandhu, M.A. (E. 1918); William

harp (E. 1879); Charles William Shaw (E. 1909); Thomas Clive Sheppard (E. 1892); Percy Slater (E. 1936); Robert Watson Clark Smith (E. 1930); Percival Stevens (E. 1885); John Wibberley (E. 1906); Ernest Williams (E. 1889); Llewelyn George Henry Wynn-Williams, B.Sc. (E. 1928). Deaths.

Associate (1).—*Col.* John Gibson Fleming, C.B.E., D.S.O., R.E. (E. 1910).

Students (2).—Charles Robert Hart (A. 1934); Leslie Ernest Turner, B.Sc. (A. 1934).

There has been one erasure under By-law 22. Erasure.

The following resignations have been received :— Resignations

Members (14).—Henry Farre Bowen (E. 1906. T. 1920); John Bowman (E. 1900. T. 1921); Theobald Stuart Butler (E. 1910. T. 1926); *Rai Bahadur* Wazir Chand Chopra (E. 1920); Edward Meirion Vivian Davies (E. 1918. T. 1933); Claude Vyvian Armit Espeut (E. 1927); Arthur Cecil Fellows (E. 1910); Alaric Hope (E. 1899. T. 1908); John Turnbull McIntyre (E. 1897. T. 1904); Bertram Grant Meaden (E. 1906. T. 1928); Charles Hubert Reynolds (E. 1899. T. 1924); Frederic Shelford, B.Sc. (E. 1897. T. 1902); Ebenezer Thomas Ward, C.B.E. (E. 1904. T. 1924); Horace Frank Waters, M.C., V.D. (E. 1908. T. 1929).

Associate Members (44).—Frederick Rhodes Armitage, B.A. (E. 1893); Leonard Ludovic Baldwin (E. 1897); Reginald John Baumgartner (E. 1911); Harold Storr Best (E. 1908); Alan Grant Birch, D.S.O., B.A. (E. 1909); Arthur Boyd, D.Sc., B.Sc., B.E. (E. 1919); Archibald Carmichael (E. 1907); Frederick William Clarke (E. 1899); Geoffrey Clarke (E. 1909); Thomas Cockrill (E. 1894); Harvey Collingridge, B.Sc. (E. 1898); William Emmanuel Macbryde Curnock, M.Sc., B.Eng. (E. 1920); Frederick George Davis (E. 1906); Ringrose Charles Willington Drew, B.A., B.A.I. (E. 1920); James Rammell Easton (E. 1891); Idris Evans (E. 1927); Gerald Roylance Chichester Fairlie (E. 1927); Frank Furnivall (E. 1898); Sydney Harold Garnett (E. 1899); Laurence Thornewaite Grace, M.C. (E. 1910); Cecil Edward Harvey (E. 1919); Ronald Victor Hitchcock, B.Sc. (E. 1911); Edwin Human (E. 1892); Charles Hilton Hutchinson, B.A. (E. 1908); Albert Edward Langley (E. 1908); John Alexander McCrindle, B.Sc. (E. 1908); Reginald George McLaughlin (E. 1906); William McNeill (E. 1893); Patrick Malone (E. 1913); Thomas John McLuckie Moore (E. 1919); George Paxton Napier (E. 1908); Frederick James Paice (E. 1906); Frank Longden Pasley (E. 1931); Alfred Leslie Polson, M.C., B.E. (E. 1917); Walter Bishop Purser (E. 1899); George Strafford Robertson, B.A., B.A.I. (E. 1912); Joseph Temple Robinson (E. 1924); Richard Napier Rutherfurd, B.Sc. (E. 1928); Joseph Shepherd (E. 1903); Jawand Singh, B.Sc. (E. 1919); Edward Speechly (E. 1909); Henry James Seaton Wade (E. 1894); John Lionel Wells (E. 1927); Rupert Hemsley Wheatley, M.A. (E. 1925).

Associate (1).—*Flight-Lieut.* William Leigh Houlbrook, R.A.F. (E. 1934) (*since elected Associate Member*).

Students (20).—Benjamin Frank Barber (A. 1928); Robert Charles Biggs (A. 1932); James Bottomley Bradley, B.A. (A. 1933); John Broadbent (A. 1930); John Richard Burgess (A. 1929); Maurice George Burges-Short (A. 1929); Gordon McMillan Burrows (A. 1929); Reginald James Dalziel Gray (A. 1929); Alexander Edgar Haller Griffits (A. 1929); John Leslie Hobson, B.Sc. (A. 1933); Fred Jepson (A. 1933); Francisco Joaquim Maria Jorge, B.Sc. (A. 1933); Neil Arbuckle McNeil (A. 1931); Laurence Francis Brooke Reeves (A. 1927); Richard John Kesterton Relph (A. 1929); Walter Paul Shewell (A. 1928); Vernon Watkins Sparrow (A. 1929); Ta-Hsung Tung, B.Sc. (A. 1935); Edwin Peter Wilders, B.Sc. (A. 1932); Norman Henry George Young, B.Sc. (A. 1934).

[APPENDIX.

APPEN

BALANCE SHEET

	£	s.	d.	£	s.	d.
To Institution Capital Account and Building Fund, *as detailed on pages 238 and 239* .	..			416,233	12	10
,, Loan on Security of Institution Buildings—						
As per last account	5,445	1	0			
Less repaid during year	1,740	0	3			
				3,705	0	9
,, Creditors	..			116	13	9
,, Sundry Credit Items in Suspense . . .	..			439	4	6
,, Repairs and Renewals Reserve, *as detailed on pages 238 and 239*	..			6,103	4	1
,, Research Reserve—*as detailed on pages 244 and 245*	..			2,206	11	7
,, W. A. P. Tait Legacy, including unspent Income	..			648	3	4
,, Sea-Action Committee Account—*as detailed on pages 244 and 245*	..			1,947	13	0
,, Trust Funds, Capital and Income Accounts—						
Capital Accounts—*as detailed on pages 242 and 243, invested per contra*	37,557	6	7			
Income Accounts—Balances unexpended—*as detailed on pages 244 and 245* . .	2,438	11	6			
				39,995	18	1
,, Institution Revenue in Suspense—						
Proportion of 1937 Subscriptions applicable to the nine months from 1st April to 31st December, 1937	15,056	17	0			
Subscriptions received in advance . . .	79	19	0			
				15,136	16	0
,, General and Contingency Reserve (being accumulations of Income over Expenditure from 1st April, 1935) *as per last account*	1,296	19	5			
Less Debit Balance transferred from this year's Revenue account (*see page 241*) .	683	19	4			
				613	0	1
				£487,145	18	0

AUDIT

We have audited the above Balance Sheet dated 31st March, 1937, and have obta
is properly drawn up so as to exhibit a true and correct view of the state of The Institut
shown by the books of The Institution.

London, 3rd May, 1937.

DIX.

31st MARCH, 1937.

			£ s. d.	£ s. d.
By Expenditure on Institution Building, including Cost of Site, *as per last account*			352,072 7 2	
Add Expenditure during the Year on Account of the Completion of the Building			9,099 13 10	361,172 1 0
By Institution Investments (including those held in respect of Repairs and Renewals Reserve) at cost, *as detailed on page 246*				54,333 11 1
Note.—*The value of these Investments at ruling prices on 31st March, 1937, amounted approximately to £52,461.*				
,, W. A. P. Tait Legacy—				
£505 19*s.* 9*d.* 3½% War Loan at cost . .			514 11 8	
Cash at Bank			133 11 8	648 3 4
Note.—*The value of the Investment at ruling price on the 31st March, 1937, amounted approximately to £518.*				
,, Sea-Action Committee Account—				
Cash at Bank				1,947 13 0
,, Trust Funds Investments, etc.—				
Capital :—				
Investments, *as detailed on pages 242 and 243*			£37,557 6 7	
Unexpended Income :—				
Investments, *as detailed on page 243*		£214 8 2		
Cash at Bank—				
On Deposit a/c	1,950 0 0			
,, Current a/c	274 3 4	2,224 3 4	2,438 11 6	39,995 18 1
,, Debtors				1,475 19 11
,, Cash at Bank and in Hand—				
At Bank—Deposit and Current Accounts—				
Institution Funds			11,641 17 5	
Building Fund			9,685 15 11	
Secretary's Drawing Account			95 5 5	
			21,422 18 9	
In Hand			69 15 0	21,492 13 9
Publications Account—				
Balance overspent, *per page 238*				6,079 17 10
Note.—*No value has been attached, for the purpose of this Balance Sheet, to the Books, Furniture, Pictures, Sculpture, etc., belonging to The Institution.*				
				£487,145 18 0

H. H. Jeffcott, *Secretary.*

RT.

information and explanations we have required. In our opinion such Balance Sheet according to the best of our information and the explanations given to us, and as

Percival D. Griffiths, F.C.A.
E. W. Monkhouse, M. Inst. C.E. } Auditors.

INSTITUTION CAPITAL ACCOUNT

	£	s.	d.
To Balance *carried down*	416,233	12	10
	£416,233	12	10

RESERVE FOR REPAIRS AND RENEWALS TO

	£	s.	d.
To Expenditure during the Year	1,332	0	6
" Balance *carried down*	6,103	4	1
	£7,435	4	7

PUBLICATIONS

	£	s.	d.	£	s.	d.
To Balance—*per last account*				2,414	12	3
" Expenditure during the Year—						
Journal	10,926	15	5			
Minutes of Proceedings	1,742	19	7			
Charters, By-laws and Lists of Members	795	19	4			
Engineering Abstracts	1,705	2	7			
Salaries, Clerical Pay and Pensions Premiums	2,988	2	4			
	18,158	19	3			
Less Credits for Advertisements, Sales, Contributions, etc.	4,493	13	8			
				13,665	5	7
				£16,079	17	10
To Balance *brought down—as per Balance Sheet, page 237*				£6,079	17	10

AND BUILDING FUND, 31ST MARCH, 1937.

	£	s.	d.
By BALANCE—*per last account*	413,103	13	10
,, BALANCE OF LEGACY UNDER THE WILL OF SIR DUGALD CLERK, M. Inst. C.E.	3,129	19	0
	£416,233	12	10
By BALANCE *brought down—as per Balance Sheet, page 236* .	£416,233	12	10

STRUCTURE, FURNITURE, FITTINGS AND MACHINERY.

	£	s.	d.
By BALANCE—*per last account*	6,028	17	7
,, INSTITUTION REVENUE ACCOUNT—Amount provided for the year—*per page 240*	1,000	0	0
,, INTEREST ON INVESTMENTS	204	14	8
,, INCOME TAX REFUNDED	28	1	6
,, PROFIT ON SALE OF INVESTMENT	173	10	10
	£7,435	4	7
By BALANCE *brought down—as per Balance Sheet, page 236* .	£6,103	4	1

ACCOUNT

	£	s.	d.
By INSTITUTION REVENUE ACCOUNT—Amount provided for the year—*per page 240*	10,000	0	0
,, BALANCE, *carried down* (being Excess of net Expenditure over Provision)	6,079	17	10
	£16,079	17	10

INSTITUTION REVENUE ACCOUNT

EXPENDITURE.

1935-36 £		£ s. d.	£ s. d.
	To HOUSE AND ESTABLISHMENT CHARGES—		
	Rates, Health, Unemployment and other Insurances	6,028 13 2	
	Electric Lighting and Power, Water-Supply, Warming, Ventilating and Telephone	892 0 0	
	Cleaning and Household Expenses	1,090 10 2	
	Refreshments and Assistance at Meetings	165 8 1	
7,930			8,176 11 5
	,, REPAIRS AND RENEWALS RESERVE—		
1,000	Amount provided for the year, *per page 239*	..	1,000 0 0
	,, SALARIES, WAGES AND RETIRING ALLOWANCES—		
	Salaries	3,531 5 0	
	Retiring Allowances	1,605 0 0	
	Clerks, Messengers and Housekeeper	5,053 0 10	
10,239			10,189 5 10
	,, PREMIUMS ON POLICIES FOR STAFF PENSIONS—		
1,383	Portion paid by the Institution	..	1,368 9 10
	,, STATIONERY, POSTAGES, ETC.—		
	Stationery and Printing	1,125 19 7	
	Postages, Telegrams and Parcels	885 13 2	
2,090			2,011 12 9
	,, PUBLICATIONS ACCOUNT—		
10,000	Amount provided for the year, *per page 239*	..	10,000 0 0
	,, RESEARCH RESERVE—		
1,000	Amount provided for the year, *per page 245*	..	2,600 0 0
	,, LIBRARY—		
	Books and Periodicals	449 19 9	
	Binding	153 2 2	
	Clerical Pay and Pensions Premiums	937 4 5	
1,579			1,540 6 4
	,, EXAMINATION EXPENSES—		
	Examiners, Printing and General	1,902 16 2	
	Salaries, Clerical Pay and Pensions Premiums	1,585 11 0	
	Postages	117 13 0	
3,483			3,606 0 2
1,255	,, CONVERSAZIONE AND ANNUAL DINNER	..	1,217 9 1
	,, DIPLOMAS AND MEDALS—		
	Diplomas	64 1 8	
	Replica of Medal	2 10 0	
41			66 11 8
	,, LOCAL ASSOCIATIONS—		
1,178	Grants to Local Associations, etc.	..	1,223 17 6
27	,, CONTRIBUTIONS TOWARDS ADVISORY COMMITTEES IN THE DOMINIONS	..	100 0 0
	,, GRANTS AND CONTRIBUTIONS—		
	Ewing Medal Fund	300 0 0	
	Congress of the International Association for Testing Materials	100 0 0	
	Parliamentary Science Committee	21 0 0	
	Engineering Joint Council	12 10 0	
	Chemical Engineering Congress	10 10 0	
	Westminster Hospital	10 10 0	
	World Power Conference	3 3 0	
186			457 13 0
	,, LEGAL AND OTHER PROFESSIONAL CHARGES—		
	Legal Charges	34 9 10	
	Audit and Accountancy Fees	183 15 0	
772			218 4 10
	,, TRAVELLING EXPENSES TO COMMITTEES	..	134 17 0
	,, MEMORIAL SERVICE, ADDRESSES, ETC.	..	23 6 0
234	,, INTEREST ON LOAN	..	160 10 9
£42,618			£44,094 16 2

FROM 1st APRIL, 1936, TO 31st MARCH, 1937.

INCOME.

	£	s.	d.	£	s.	d.	1935–36 £
By Subscriptions applicable to the Financial Year 1936–1937	..			32,904	15	1	*31,992*
,, Entrance Fees	..			5,441	10	0	*6,039*
,, Life Composition	..			100	0	0	
,, Interest, Dividends, etc.—							
On Institution Investments	1,312	7	1				
On Deposit and Current Accounts . . .	111	13	1				
Income Tax refunded for the year 1935–6 .	203	0	8				
				1,627	0	10	*2,337*
,, Examination Fees	..			3,272	19	11	*3,290*
,, Library Fund Donations	..			64	11	0	*49*
,, *Rent of 1, Great George Street (now demolished)*	..						*208*
				43,410	16	10	*43,915*
,, Balance, being Excess of Expenditure over Income for the Year carried to General and Contingency Reserve, *page 236*	..			683	19	4	*1,297* (Cr.)
				£44,094	16	2	*42,618*

TRUST

Capital Accounts and Investment thereof and Investment

Capital Accounts. £	s.	d.		Investments. Capital. £	s.	d.	Unexpended Income. £	s.	d.
8,038	9	4	Telford Fund.						
			£8,738 13*s*. 0*d*. 2½% Consols	7,988	9	4			
			£50 16*s*. 11*d*. 3½% War Loan	50	0	0			
270	0	0	Manby Donation.						
			£250 London & North-Eastern Railway 4% 2nd Guaranteed Stock . .	270	0	0			
6,337	12	4	Miller Fund.						
			£5,129 17*s*. 5*d*. 2½% Consols	4,850	2	4			
			£1,513 15*s*. 9*d*. 3½% War Loan	1,487	10	0			
500	0	0	Howard Bequest.						
			£352 11*s*. 5*d*. 2½% Consols . Cost of Medal Die . . .	500	0	0			
600	0	0	Trevithick Memorial.						
			£103 2½% Consols . . .	100	0	0			
			£506 5*s*. 7*d*. 3½% Conversion Loan 1961	500	0	0			
540	0	0	Crampton Bequest.						
			£512 15*s*. 11*d*. 2½% Consols	500	0	0			
			£40 13*s*. 7*d*. 3½% War Loan	40	0	0			
1,234	14	0	James Forrest Lecture and Medal Fund.						
			£465 Southern Railway 4% Debenture Stock . . .	604	14	0			
			£667 5*s*. 8*d*. 3½% War Loan	630	0	0			
1,530	18	0	Palmer Scholarship.						
			£1,496 6*s*. 1*d*. Metropolitan 3% Consolidated Stock .	1,430	18	0			
			£100 9*s*. 8*d*. 3½% War Loan	100	0	0			
1,080	0	0	John Bayliss Bequest.						
			£1,013 17*s*. 10*d*. London County 3% Stock 1920 .	1,000	0	0			
			£80 7*s*. 10*d*. 3½% War Loan	80	0	0			
1,318	11	8	The Indian Fund.						
			£1,353 4*s*. 2*d*. 2½% Consols .	1,148	11	8			
			£171 13*s*. 3*d*. 3½% War Loan	170	0	0			
1,000	0	0	Vernon-Harcourt Bequest.						
			£1,082 9*s*. 10*d*. London County 3% Stock 1920 .	1,000	0	0			
22,450	5	4	*Carried forward* . .	22,450	5	4			

FUNDS.

OF UNEXPENDED INCOME AT 31ST MARCH, 1937.

Capital Accounts.				Investments. Capital.			Investments. Unexpended Income.		
£	s.	d.		£	s.	d.	£	s.	d.
22,450	5	4	*Brought forward* . . .	22,450	5	4			
1,300	0	0	WEBB BEQUEST.						
			£1,055 7s. 2d. Metropolitan Water Board 3% "B" Stock	1,000	0	0			
			£303 16s. 2d. 3½% War Loan	300	0	0			
2,733	1	10	WILLIAM LINDLEY FUND.						
			£1,214 London Midland & Scottish Railway 4% Debenture Stock . . .	1,584	16	8			
			£1,109 2s. 9d. 4½% India Stock, 1950–1955 . . .	1,005	15	2			
			£151 9s. 7d. 3½% War Loan	142	10	0			
725	0	0	KELVIN MEDAL FUND.						
			£757 18s. 11d. 3½% War Loan	725	0	0			
4,250	0	0	CHARLES HAWKSLEY BEQUEST.						
			£955 Metropolitan Water Board 3% "B" Stock .	573	0	0			
			£500 South Essex Waterworks 5% Preference Stock	435	0	0			
			£70 5% Sheffield Corporation Water Annuities . . . £40 4% Sheffield Corporation Water Annuities . .	1,992	0	0			
			£1,257 17s. 4d. 3½% War Loan	1,250	0	0			
1,101	6	5	COOPERS HILL WAR MEMORIAL.						
			£1,239 15s. 0d. 3½% War Loan	1,101	6	5			
			£215 12s. 3d. 3½% War Loan	..			214	8	2
3,570	4	0	C. C. LINDSAY CIVIL ENGINEERING SCHOLARSHIP FUND.						
			£3,500 3½% War Loan . .	3,570	4	0			
320	0	0	BAKER MEDAL FUND. . .						
			£290 5s. 8d. London County Cons. 4½% Stock 1945–85	320	0	0			
807	9	0	JAMES ALFRED EWING MEDAL FUND.						
			£798 12s. 9d. Middlesex County Council 3% Stock 1961–1966	807	9	0			
300	0	0	G. H. DENNISON FUND.						
			£305 12s. 9d. 3% Local Loans	300	0	0			
37,557	6	7	*As per Balance Sheet, page 237*	*37,557	6	7	*214	8	2

NOTE.—* *The value of these Investments at ruling prices on 31st March, 1937, amounted approximately to £35,965 and £221 respectively.*

TRUST FUNDS INCOME ACCOUNTS FROM

Trust Fund.	Balance at 1st April, 1936. £	s.	d.
Telford Fund	52	2	0
Manby Fund	20	5	2
Miller Fund	177	5	11
Howard Bequest	45	5	10
Trevithick Memorial	13	19	0
Crampton Bequest	4	2	11
James Forrest Lecture and Medal Fund	26	8	0
Palmer Scholarship Fund	29	15	8
John Bayliss Bequest	46	17	0
Indian Fund	26	19	2
Vernon-Harcourt Bequest	124	12	1
Webb Bequest	152	1	7
William Lindley Fund	479	18	5
Kelvin Medal Fund	70	19	8
Charles Hawksley Bequest	111	5	3
Coopers Hill War Memorial Fund	322	6	10
C.C. Lindsay Civil Engineering Scholarship Fund	217	9	9
Baker Medal Fund	17	9	6
James Alfred Ewing Medal Fund	0	0	0
G. H. Dennison Fund	0	0	0
Totals	1,939	3	9

COMMITTEE ON THE DETERIORATION OF

Account from 1st April, 1936,

	£	s.	d.
To Amount paid on behalf of or to the Committee during the year to 31st March, 1937	840	9	6
,, Balance *carried down*	1,947	13	0
	£2,788	2	6

RESEARCH

	£	s.	d.	£	s.	d.
To Research—						
Vibrated Concrete Research	400	0	0			
Pile Driving Research	312	10	0			
Special Cements Research	100	0	0			
Earth Pressures Research	250	0	0			
				1062	10	0
To Grants to other Bodies—						
Grant to Froude Tank Research Fund	100	0	0			
Joint Committee on Materials	10	0	0			
				110	0	0
To Administration Expenses—						
Travelling Expenses of Committees	73	8	3			
Salaries and Clerical Pay	613	15	0			
Sundries	22	14	2			
				709	17	5
				1,882	7	5
To Balance *carried down*				2,206	11	7
				£1,088	19	0

1st APRIL, 1936, TO 31st MARCH, 1937.

Income: Including Income Tax refunded for the year 1935–1936.			Expenditure on Scholarships, Prizes, Lectures, etc.			Balance at 31st March, 1937.		
£	s.	d.	£	s.	d.	£	s.	d.
220	10	4	161	0	9	111	11	7
9	19	8	21	9	0	8	15	10
182	2	11	72	14	9	286	14	1
9	1	0	0	0	0	54	6	10
20	5	3	10	15	0	23	9	3
14	4	11	15	0	0	3	7	10
41	17	2	47	9	6	20	15	8
48	11	1	40	0	0	38	6	9
33	1	10	30	0	0	49	18	10
39	19	4	35	0	0	31	18	6
32	13	9	63	0	0	94	5	10
42	13	3	0	0	0	194	14	10
104	19	9	0	0	0	584	18	2
26	17	9	0	0	0	97	17	5
206	4	3	125	0	0	192	9	6
51	9	8	46	7	11	327	8	7
123	12	1	67	10	0	273	11	10
13	1	3	0	0	0	30	10	9
11	14	6	0	0	0	11	14	6
1	14	11	0	0	0	1	14	11
1,234	14	8	735	6	11	†2,438	11	6
						As per Balance Sheet p. 236.		

† Of which £214 8*s*. 2*d*. is invested (*see page 243*).

STRUCTURES EXPOSED TO SEA ACTION.

TO 31st MARCH, 1937.

	£	s.	d.
By Balance, as per last Account	2,121	18	3
,, Subscriptions	655	14	8
,, Interest on Deposit	10	9	7
	£2,788	2	6
,, Balance *brought down as per Balance Sheet, page 236*	£1,947	13	0

RESERVE

	£	s.	d.
By Balance, as per last Account	1,200	0	0
,, Contributions from other Bodies	288	19	0
,, Institution Revenue Account—Amount provided for the year—*per page 240*	2,600	0	0
	£4,088	19	0
Balance *brought down as per Balance Sheet, page 236*	£2,206	11	7

INSTITUTION INVESTMENTS AT 31st MARCH, 1937 (INCLUDING THOSE HELD IN RESPECT OF REPAIRS AND RENEWALS RESERVE) AT COST.

£	s.	d.		£	s.	d.
3,000	0	0	Metropolitan Water Board 3% "B" Stock .	2,958	16	0
6,000	0	0	London and North Eastern Railway 4% Debenture Stock	7,749	18	3
6,000	0	0	London Midland and Scottish Railway 4% Debenture Stock	7,452	14	8
2,545	0	0	London Midland and Scottish Railway 4% Guaranteed Stock	1,976	7	10
9,905	18	7	3½% War Loan	7,626	2	1
2,720	5	5	London Passenger Transport Board 4½% "A" Stock	3,327	9	3
3,809	0	2	3½% War Loan	3,824	8	6
452	0	0	London Midland and Scottish Railway 4% Guaranteed Stock	351	3	7
989	14	7	London Passenger Transport Board 4½% "A" Stock	1,210	12	11
5,327	6	5	New Zealand 3% Stock 1952-1955	5,334	7	6
9,400	0	0	Middlesex County Council 3% Redeemable Stock 1961-1966	9,391	11	6
470	4	3	3% Local Loans	461	3	0
			National Gas and Oil Engine Co., Ltd., 3,336 Ordinary Shares of £1	2,668	16	0
			As per Balance Sheet, page 237	£54,333	11	1

NOTE.—*The value of these Investments at ruling prices on 31st March, 1937, amounted approximately to £52,461*

Paper No. 5069.

"The Open-Frame Girder."[1]

By Gerald Salmon Gough, M.A., Assoc. M. Inst. C.E.

TABLE OF CONTENTS.

Introduction.

The greater part of this Paper deals with the problem of the uniform open-frame girder, which Mr. E. H. Bateman solved in his recent Paper.[2] The method of analysis now used is based on the same assumptions and leads to results which are numerically identical, but the final solution is expressed in another form which has greater physical significance and which can be more readily applied to any girder, of any number of bays and of any constant ratio of stiffness between verticals and chords. Further, a slight modification of procedure gives a method of obtaining an approximation to the moment-distribution in a girder in which the verticals are not of equal cross-sections. These reasons seem sufficient to justify a further Paper on the same subject.

General Method of Analysis.

The girder is assumed to be uniform; that is, the chords are parallel and of constant panel-length and cross-section, and the verticals are all equal. The stiffness of the verticals is not necessarily equal to that of the chord-panels. The analysis first considers

[1] Correspondence on this Paper can be accepted until the 1st September, 1937, and will be published in the Institution Journal for October, 1937.—Sec. Inst. C.E.

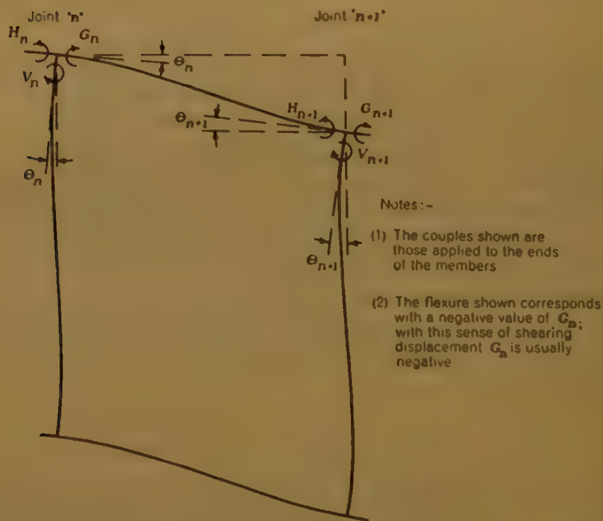
[2] "The Open-Frame Girder," Journal Inst. C.E., vol. 1 (1935–36), p. 67. (November, 1935.)

the moment-distribution in a girder of this description and of infinite length, the load and its reactions being applied locally. The local load-systems are then superposed to give an expression for a general loading. Finally the necessary modification for finite length is obtained by a method of reflection.

The complete distribution is reached by way of the bending moments at the mid-points of the chord-panels. With the panel shearing forces, these "mid-panel moments" give the bending moments at the ends of the chord-panels and at the ends of the verticals. This is an alternative to the use of the points of inflexion of the chord-panels. The present method is believed to be preferable, as it shows that these moments are directly related to the loads in their neighbourhood, rather than to the shearing forces or bending moments. In this way some conception is obtained of the mechanics of the open-frame girder.

NOTATION. (*Fig. 1.*)

Fig. 1.



Let L denote the panel-length.

" K_c " "stiffness" of a chord-panel, that is, its moment of inertia divided by its length.

" K_v " "stiffness" of a vertical.

" W " load applied at a panel-point, positive for a downward load, negative for a reaction.

Let F denote the total shearing force between two panel-points, positive when the "sagging" bending moment increases towards the right.
" M " total "sagging" bending moment.
" N " "sagging" bending moment at the mid-point of a chord-panel, "the mid-panel moment."
" G " "sagging" bending moment at the left-hand end of a chord-panel.
" H " "sagging" bending moment at the right-hand end of a chord-panel.
" V " clockwise couple applied to the end of a vertical.
" θ " the clockwise rotation of a joint.
" s " stiffness-ratio, K_v/K_c.
" r " series ratio $= 3s + 1 - \sqrt{3s(3s+2)}$. (This is explained on p. 251.)
" m and n be subscripts denoting specific panel-points, thus:—

$M_n =$ the total bending moment at panel-point "n,"

$N_{n,\,n+1} =$ the mid-panel moment between panel-points "n" and "$n+1$."

General Equations for Mid-Panel Moments.

In frames of this nature the deformations of the members due to bending are much larger than those due to direct tension or compression, and it is justifiable to neglect the effects of the latter on the distribution of bending moment. With this approximation, it is clear that the deformations of the two chords are identical, and, similarly, that the shearing forces and bending moments in these chords are respectively identical. In particular, the total shearing force is divided equally between the chords, and the couples at the upper and lower ends of the verticals are equal.

Using the notation already set out, considerations of equilibrium give, for the external forces:—

$$W_n = F_{n-1,\,n} - F_{n,\,n+1} \quad \ldots\ldots \quad (1)$$

$$L \,.\, F_{n,\,n+1} = M_{n+1} - M_n \quad \ldots\ldots \quad (2)$$

Also for any chord-panel (*Fig. 2*, p. 250),

$$G_n = N_{n,\,n+1} - \frac{L}{4} F_{n,\,n+1} \quad \ldots\ldots \quad (3)$$

$$H_{n+1} = N_{n,\,n+1} + \frac{L}{4} F_{n,\,n+1} \quad \ldots\ldots \quad (4)$$

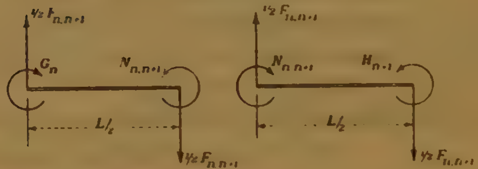
and for any joint,

$$V_n = H_n - G_n \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

Further, the flexural conditions for the chords give:

$$\begin{aligned} \theta_n - \theta_{n+1} &= \text{the relative rotation of the ends} \\ &= \text{area of the bending-moment diagram divided by } EI \\ &= \frac{1}{2EK_v}(G_n + H_{n+1}) \\ &= \frac{N_{n,\,n+1}}{EK_c}, \text{ using equations (3) and (4).} \end{aligned}$$

Fig. 2.



For the verticals, the flexural relation is:

$$\theta_n = \frac{V_n}{6EK_v}.$$

Eliminating the rotations θ_n, θ_{n+1}, and substituting $K_v/K_c = s$,

$$V_n - V_{n+1} = 6sN_{n,\,n+1} \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

and substituting from equation (5),

$$H_n - G_n - H_{n+1} + G_{n+1} = 6sN_{n,\,n+1}\,.$$

Finally, by substitution from equations (3) and (4),

$$\begin{aligned} N_{n-1,\,n} - 2(3s + 1)N_{n,\,n+1} + N_{n+1,\,n+2} \\ = \frac{L}{4}(F_{n+1,\,n+2} - F_{n-1,\,n}) \quad . \quad (7) \end{aligned}$$

This is the fundamental equation upon which the solution depends.

Girder of Infinite Length.

Local Load.—Now consider a girder of infinite length, and suppose that it carries a single load at joint "m," and that this load is supported by equal reactions at the neighbouring joints "$m - 1$" and "$m + 1$." All other joints are taken to be free from load.

Let the load be such that the bending moment at "m" is M_m. The shearing forces will then be given by:

$$L \,.\, F_{m-1,\,m} = M_m \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

and

$$L \,.\, F_{m,\,m+1} = -M_m \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

In all other panels the shearing force is zero.

In the unloaded panels equation (7) reduces to

$$N_{n-1,\,n} - 2(3s+1)N_{n,\,n+1} + N_{n+1,\,n+2} = 0.$$

Clearly the series of equations, of which this is typical, is satisfied by the geometrical progression

$$\begin{aligned} N_{n-1,\,n} &= N \\ N_{n,\,n+1} &= \rho N \\ N_{n+1,\,n+2} &= \rho^2 N, \text{ etc.,} \end{aligned}$$

provided that ρ satisfies the equation

$$\rho^2 - 2(3s+1)\rho + 1 = 0,$$

which gives

$$\rho = 3s + 1 \pm \sqrt{3s(3s+2)}.$$

The two roots are mutually reciprocal, so that the smaller is less than unity. Both roots are always real and positive for positive values of s. Denoting the smaller root by r,

$$r = 3s + 1 - \sqrt{3s(3s+2)}$$

and
$$2(3s+1) = \frac{1}{r} + r \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

The two values of ρ give two series of moments in geometrical progression, either of which might be applicable, or both simultaneously. Physical considerations, however, show that one series only is possible on one side of the local load, and the other only on the opposite side, as the mid-panel moments must decrease indefinitely when proceeding outwards from the load. It follows that the series with the smaller root, r, as common ratio must apply on the right-hand side of the load, and the series with ratio $\frac{1}{r}$ be applicable on the left-hand side. The two series, in fact, form a symmetrical arrangement.

In the region of the load, equation (7) gives, on substituting, in turn, $n = m - 3, m - 2, \ldots m + 2$, and using equations (8), (9) and (10),

$$\begin{aligned}
N_{m-4,\,m-3} - \left(\frac{1}{r} + r\right)N_{m-3,\,m-2} + N_{m-2,\,m-1} &= 0, \\
N_{m-3,\,m-2} - \left(\frac{1}{r} + r\right)N_{m-2,\,m-1} + N_{m-1,\,m} &= \frac{M_m}{4}, \\
N_{m-2,\,m-1} - \left(\frac{1}{r} + r\right)N_{m-1,\,m} + N_{m,\,m+1} &= -\frac{M_m}{4}, \\
N_{m-1,\,m} - \left(\frac{1}{r} + r\right)N_{m,\,m+1} + N_{m+1,\,m+2} &= -\frac{M_m}{4},
\end{aligned}$$

$$N_{m,\,m+1} - \left(\frac{1}{r}+r\right)N_{m+1,\,m+2} + N_{m+2,\,m+3} = \frac{M_m}{4},$$

$$N_{m+1,\,m+2} - \left(\frac{1}{r}+r\right)N_{m+2,\,m+3} + N_{m+3,\,m+4} = 0,$$

Utilizing the above considerations, it is permissible to put:

$$\begin{aligned}
N_{m-1,\,m} &= N_{m,\,m+1},\\
N_{m-2,\,m-1} &= N_{m+1,\,m+2},\\
N_{m-3,\,m-2} &= N_{m+2,\,m+3} = rN_{m+1,\,m+2},\\
N_{m-4,\,m-3} &= N_{m+3,\,m+4} = r^2N_{m+1,\,m+2}, \text{ etc.}
\end{aligned}$$

Then the first and last equations are identically satisfied, and the others, being identical in pairs, lead to two equations which are:

$$\left(\frac{1}{r}-1+r\right)N_{m,\,m+1} + N_{m+1,\,m+2} = \frac{M_m}{4}$$

and

$$N_{m,\,m+1} - \frac{1}{r}N_{m+1,\,m+2} = \frac{M_m}{4}.$$

The solution of these is:

$$N_{m,\,m+1} = r\frac{M_m}{4},$$

$$N_{m+1,\,m+2} = -r(1-r)\frac{M_m}{4};$$

and consequently

$$N_{m+2,\,m+3} = -r^2(1-r)\frac{M_m}{4},$$

$$N_{m+3,\,m+4} = -r^3(1-r)\frac{M_m}{4}, \quad \text{etc.}$$

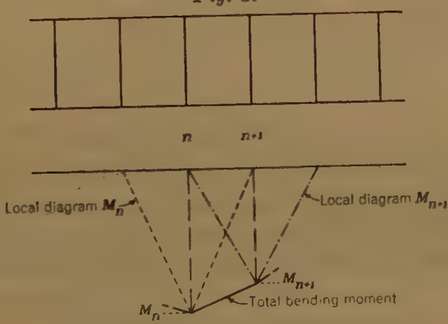
On the left-hand side of joint "m" the same series of moments appears in reverse order.

Any Load-System.—By superposition of loads such as that already considered, any bending-moment diagram can be built up. It will be clear from *Fig. 3* that direct addition of a set of local diagrams with maximum ordinates of M_m, M_{m+1}, M_{m+2}, etc., gives a total bending-moment diagram with the same ordinates at the panel-points. Hence, in order to obtain the mid-panel moments corresponding to any bending-moment diagram, it is only necessary to add a series of sets of moments, such as that obtained in the preceding paragraph.

The scheme of the addition is given in the following Table :—

Panel-point moment.	Coefficients of $\frac{M}{4}$ in series for mid-panel moments.				
	$N_{n-2,\,n-1}$	$N_{n-1,\,n}$	$N_{n,\,n+1}$	$N_{n+1,\,n+2}$	$N_{n+2,\,n+3}$
M_{n-2}	r	$-r(1-r)$	$-r^2(1-r)$	$-r^3(1-r)$	$-r^4(1-r)$
M_{n-1}	r	r	$-r(1-r)$	$-r^2(1-r)$	$-r^3(1-r)$
M_n	$-r(1-r)$	r	r	$-r(1-r)$	$-r^2(1-r)$
M_{n+1}	$-r^2(1-r)$	$-r(1-r)$	r	r	$-r(1-r)$
M_{n+2}	$-r^3(1-r)$	$-r^2(1-r)$	$-r(1-r)$	r	r
M_{n+3}	$-r^4(1-r)$	$-r^3(1-r)$	$-r^2(1-r)$	$-r(1-r)$	r

Fig. 3.



This gives, on re-arrangement,

$$N_{n,\,n+1} = \frac{r}{4}(M_n + M_{n+1}) - \frac{r(1-r)}{4}[(M_{n-1} + M_{n+2})$$
$$+ r(M_{n-2} + M_{n+3}) + r^2(M_{n-3} + M_{n+4}) + \text{etc.}] \quad . \quad (11)$$

This equation expresses the mid-panel moment in terms of the panel-point moments. It is a complete solution in itself, but a more convenient form is obtained by conversion to an expression in terms of the panel-point loads.

From equation (2),

$$M_{n-m}+M_{n+m+1}=M_{n-m+1}-M_{n+m}-L(F_{n-m,\,n-m+1}-F_{n+m,\,n+m+1}),$$

which, by repeated application of equation (1), leads to

$$M_{n-m} + M_{n+m+1} = M_{n-m+1} + M_{n+m} - L[(W_n + W_{n+1})$$
$$+ (W_{n-1} + W_{n+2}) + \quad . \quad . \quad . \quad +(W_{n-m-1} + W_{n+m})].$$

Similar equations apply for all "moment pairs." Adding all equations of the series up to and including that above :—

$$M_{n-m} + M_{n+m+1} = M_n + M_{n+1} - L[(m-1)(W_n + W_{n+1}) + (m-2)(W_{n-1} + W_{n+2}) + (m-3)(W_{n-2} + W_{n+3}) + \dots + (W_{n-m+1} + W_{n+m})].$$

Finally, substituting in equation (11),

$$N_{n,n+1} = \frac{r}{4}(M_n + M_{n+1})[1 - (1-r)(1 + r + r^2 + r^3 + \dots)]$$
$$+ \frac{r(1-r)L}{4}[(W_n + W_{n+1})(1 + 2r + 3r^2 + 4r^3 + \dots)$$
$$+ (W_{n-1} + W_{n+2})(r + 2r^2 + 3r^3 + 4r^4 + \dots)$$
$$+ (W_{n-2} + W_{n+3})(r^2 + 2r^3 + 3r^4 + \dots) + \text{etc.}]$$
$$= \frac{rL}{4(1-r)}[(W_n + W_{n+1}) + r(W_{n-1} + W_{n+2}) + r^2(W_{n-2} + W_{n+3}) + \text{etc.}] \quad . \quad (12)$$

This is the complete solution for the mid-panel moments of the infinite girder. The series is reasonably convergent; the "load-pairs" are roughly of the same order throughout and r is unlikely to be greater than $\frac{1}{3}$, a more probable value being about $\frac{1}{5}$.*

Girder of Finite Length.

By a special interpretation of the loads W_n, etc., equation (12) can be used without modification for the mid-panel moments of the finite girder.

Consider a system of loads, such as is shown in *Fig. 4*. In this a finite set of loads, together with the appropriate reactions, is repeated indefinitely, with each repetition there being a reflection and a change of sign. Further, suppose that the shearing force and bending moment are zero in the "intervals" between each reaction and its reflection, which is obviously a possible condition. If equation (12) be applied to determine the mid-panel moment for any "interval," all the load-pairs will vanish on account of the odd-valued symmetry about the intervals. Hence for each interval :—

(*a*) the mid-panel moment is zero;

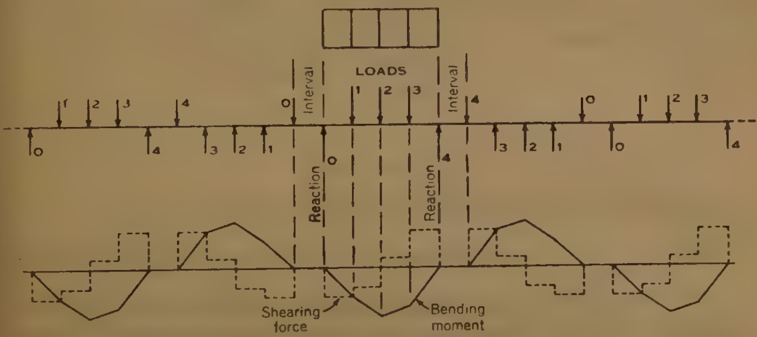
(*b*) the bending moments at the ends of the chord-panel are zero, as the mid-panel moment and the shearing force are zero;

* A later section of the Paper (p. 256) deals with the value of r.

(c) the direct forces in the chord-panel are zero, as there is no total bending moment and no longitudinal force.

Hence, in the intervals, the chords are entirely free from stress, and their removal would in no way affect the distribution of stress in the remainder of the infinite girder. Without these connecting chords the girder divides into a series of finite lengths, for each of

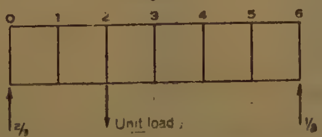
Fig. 4.



which the correct distribution of moments has been obtained. The necessary rule for the finite girder is, then: reflect the load-system, including the reactions, indefinitely, each time with a change of sign; then apply equation (12).

An example will serve to make this clear.[1] Consider a six-bay girder, with equal stiffness of chord-panels and of verticals, a single

Fig. 5.



load being applied as shown in *Fig. 5*, and suppose that it is required to determine the mid-panel moment for the end-panel (0, 1).

To the right of the panel the loads and reflections are :—

$$0,\ 1,\ 0,\ 0,\ 0,\ -\tfrac{1}{3},\ +\tfrac{1}{3},\ 0,\ 0,\ 0,\ -1,\ \text{etc.}$$

and to the left

$$-\tfrac{2}{3},\ +\tfrac{2}{3},\ 0,\ -1,\ 0,\ 0,\ 0,\ +\tfrac{1}{3},\ -\tfrac{1}{3},\ 0,\ 0,\ \text{etc.}$$

Now $s = 1$, so that $r = 0{\cdot}1270$ and $\dfrac{r}{4(1-r)} = 0{\cdot}0364.$

[1] A second and more complete example is included in the Appendix (p. 260).

Then, applying equation (12),

$$N_{0,1} = 0{\cdot}0364L[(0 - \tfrac{2}{3}) + 0{\cdot}1270(1 + \tfrac{2}{3}) + 0{\cdot}1270^2(0 + 0) + 0{\cdot}1270^3(0 - 1) + 0{\cdot}1270^4(0 + 0) + \ldots \text{ etc.}]$$
$$= -0{\cdot}0166L.$$

This agrees with Mr. Bateman's results, Table II in his Paper,[1] giving for the same position of the load

$$\alpha_1 = 1{\cdot}4663\frac{L}{8}, \qquad \beta_1 = 1{\cdot}2003\frac{L}{8},$$

whence
$$N_{0,1} = \frac{\beta_1 - \alpha_1}{2} = -0{\cdot}0166L.$$

Values of r.

In Table I, and graphically in *Fig. 6*, are shown the values of r and of $\dfrac{r}{4(1-r)}$ corresponding to values of the stiffness-ratio, $K_v/K_c(=s)$, between the limits $\frac{1}{8}$ and 8. These limits far more than cover practical conditions, but the extreme values are of some interest in showing the effect of the stiffness-ratio.

Table I.

$s = \dfrac{K_c}{K_v}$.	r.	$\dfrac{r}{4(1-r)}$.	Number of terms.	
			Two figures.	Three figures.
$\frac{1}{8}$	0·4313	0·1896	5	8
$\frac{1}{4}$	0·3139	0·1144	3	5
$\frac{1}{2}$	0·2087	0·0659	2	4
1	0·1270	0·0364	1	3
2	0·0718	0·0193	1	2
4	0·0385	0·0100	1	1
8	0·0200	0·0051	1	1

Table I has additional columns providing somewhat rough indications as to the number of terms of the series necessary to give two and three significant figures in the mid-panel moments. These columns give, in fact, the number of terms after which the coefficient $\dfrac{r}{4(1-r)} \times r^n$ is less than 0·005 or 0·0005. Towards the ends of the girder, where the moments are greater than at the centre, the maximum moments are considerably larger than the mid-panel moments, and the accuracy obtained is higher than that indicated

[1] *Loc. cit.*

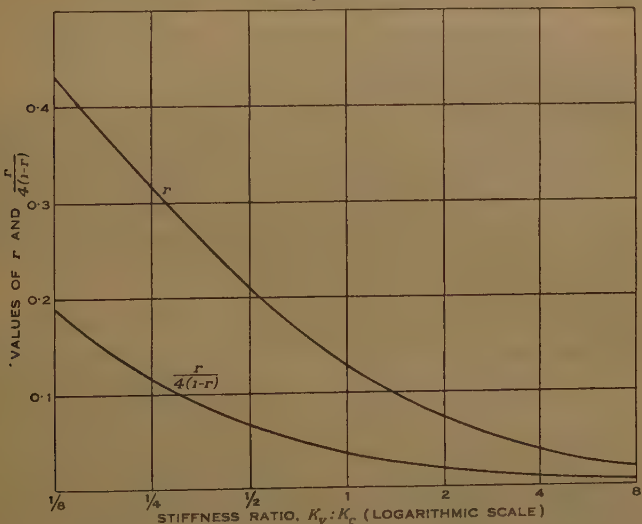
by the figures above. For most purposes the smaller number of terms will be sufficient. It will be seen that the calculation is quite handy provided that the stiffness-ratio is not less than $\frac{1}{4}$.

When this method is used for a first approximation in design an exact value of s need not be assumed, and it will be easier to work with a simple fraction for r such as $\frac{1}{3}$.

Girder with Graduated Verticals. Approximate Methods.

In practice the bending moments in the verticals near the centre of the girder will be less than those in the end verticals, so that the

Fig. 6.



central verticals can, with advantage, be made less stiff. This departure from uniformity invalidates the method of analysis used in obtaining equation (12); but if the variation in stiffness is not very large, fair estimates of the mid-panel moments can be obtained by slight modification of procedure. Two methods are available :—

(*a*) Throughout the whole girder, a single value of r is used, based on the average of the stiffness-ratios.

(*b*) Each mid-panel moment is calculated for a value of r appropriate to the mean of the stiffnesses of the verticals bounding the panel.

The mid-panel moments are not very sensitive to variation in the stiffness-ratio, provided that the latter is of the order of 1 or greater, as is shown by the lower curve of *Fig. 6*. Consequently it might

be expected that either method would give fair accuracy. Unfortunately, however, both methods neglect one effect of the graduation in stiffness. With equal stiffnesses, the mid-panel moments are zero under a uniform shearing force; with graduated stiffnesses, however, the points of inflexion of the chord-panels will move away from the stiffer joints (that is, towards the centre of the girder), and the mid-panel moments will not be zero under uniform shearing force. In general, with a shearing force changing in sense near the centre, this shift of the points of inflexion will cause negative mid-panel moments. In the centre, then, where the mid-panel moments are positive, the approximate method (*b*) will tend to give over-estimates; method (*a*) may not do so, because for the centre a value of *r* is used which is slightly too large, and this in itself will tend to underestimation. The whole behaviour, however, is too complex for generalization, and an example will be more useful to illustrate these points and to show the sort of accuracy obtainable.

Exact analysis and methods (*a*)[1] and (*b*) have been applied to a seven-panel girder having stiffness-ratios from the end to the centre of 1, 1, $\frac{5}{6}$, and $\frac{2}{3}$. Two loadings have been considered; unit loads on all panel-points, and unit loads on half the span (points 1, 2 and 3). The results are set out in Tables II and III.

TABLE II.—MAXIMUM BENDING MOMENTS IN CHORD-PANELS. (SEVEN-PANEL GIRDER.)

Panel.	Whole span loaded.			Half span loaded.		
	Exact.	(*a*)	(*b*)	Exact.	(*a*)	(*b*)
0, 1	0·806 L.	0·810 L.	0·804 L.	0·563 L.	0·565 L.	0·563 L.
1, 2	0·548	0·575	0·571	0·349	0·364	0·360
2, 3	0·335	0·344	0·360	0·131	0·124	0·137
3, 4	0·116	0·095	0·131	0·272	0·263	0·280
4, 5		(figures repeat)		0·225	0·221	0·223
5, 6				0·230	0·218	0·217
6, 7				0·243	0·245	0·241

Maximum bending moments are given in preference to the mid-panel moments, as from the practical point of view the latter tend to exaggerate the importance of the errors involved.

It will be seen that method (*b*) is, on the whole, rather more accurate than method (*a*), as might be expected. Only three of the moments obtained by method (*b*) are inaccurate by amounts of more than 5 per cent.; there is an excess of 13 per cent. in the

[1] The application of method (*a*) to this problem corresponds exactly with the example worked in the Appendix (p. 260).

TABLE III.—MAXIMUM BENDING MOMENTS IN VERTICALS.
(SEVEN-PANEL GIRDER.)

Vertical.	Whole span loaded.			Half span loaded.		
	Exact.	(*a*).	(*b*).	Exact.	(*a*).	(*b*).
0	0·806 L.	0·810 L.	0·804 L.	0·563 L.	0·565 L.	0·563 L.
1	1·145	1·116	1·125	0·730	0·714	0·720
2	0·714	0·731	0·711	0·290	0·312	0·294
3	0·218	0·248	0·228	0·141	0·139	0·143
4				0·359	0·387	0·371
5		(figures repeat)		0·424	0·419	0·417
6				0·415	0·402	0·405
7				0·243	0·245	0·241

value of the smallest moment, whilst there is also one excess of 8 per cent. and one deficiency of 6 per cent., neither on large figures. The general run of the differences is about 3 per cent., more frequently as an excess. None of these errors seems to be of any great importance, and it appears that method (*b*) is sufficiently accurate for most purposes. Method (*a*) is definitely inferior.

CONCLUSION.

It has been shown that in the uniform girder the mid-panel moments can be readily calculated from the panel-point loads. The expression obtained, equation (12), taken with the behaviour of the series ratio r, permits the following deductions :—

(1) The mid-panel moment is directly due to the loads in its neighbourhood.

(2) As the relative stiffness of the verticals increases, the mid-panel moments decrease and the effect of any load is more localized.

These effects suggest that the local sagging of the chords is responsible for the mid-panel moments. This conception may possibly help to provide a mental picture of the mechanics of the open-frame girder.

For the girder with graduated verticals, the same method may be used with slight modification. The results obtained are then only approximate and the errors are somewhat uncertain, but with moderate variation in the stiffness of the verticals no serious under-estimate of the bending moments is probable.

The Paper is accompanied by one sheet of drawings from which the Figures in the text have been prepared, and by the following Appendix.

APPENDIX.

Complete Calculations for One Loading.

Girder.—Seven panels: stiffness-ratio $\frac{5}{6}$ (constant).

Loadings.—Three panel-loads of 7 units on points 1 to 3 (7 units taken to give whole numbers for the reactions).

$$s = 0{\cdot}833$$

$$r = \tfrac{1}{7} \text{ nearly}$$

$$\frac{r}{4(1-r)} = \frac{1}{24} \qquad \frac{r^2}{4(1-r)} \text{ etc.} = \frac{1}{168}, \frac{1}{1176}, \frac{1}{8232}.$$

Table A.—Total Bending Moment.

Point.	Loads (W).	Shearing force (F).	Total bending moment (M). Panel-point.	Mid-panel.
0	− 15		0	
		+ 15		7·5
1	+ 7		15	
		+ 8		19
2	+ 7		23	
		+ 1		23·5
3	+ 7		24	
		− 6		21
4	0		18	
		− 6		15
5	0		12	
		− 6		9
6	0		6	
		− 6		3
7	− 6		0	

Table B.—Mid-Panel Moments.

Panel 0, 1.

Loads. Left.	Right.	W pair.	Coefficient.	N.
− 15	+ 7	− 8	$\frac{1}{24}$	− 0·3333
+ 15	+ 7	+ 22	$\frac{1}{168}$	+ 0·1310
− 7	+ 7	0	$\frac{1}{1176}$	0
− 7	0	− 7	$\frac{1}{8232}$	− 0·0009
			N_{01}	− 0·203

TABLE B.—*continued.*

Panel 1, 2.

Loads. Left.	Loads. Right.	*W* pair.	*N*.
+ 7	+ 7	+ 14	+ 0·5833
− 15	+ 7	− 8	− 0·0476
+ 15	0	+ 15	+ 0·0128
− 7	0	− 7	− 0·0009
		N_{12}	+ 0·548

Panel 2, 3.

Loads. Left.	Loads. Right.	*W* pair.	*N*.
+ 7	+ 7	+ 14	+ 0·5833
+ 7	0	+ 7	+ 0·0417
− 15	0	− 15	− 0·0128
+ 15	0	+ 15	+ 0·0018
		N_{23}	+ 0·614

Panel 3, 4.

Loads. Left.	Loads. Right.	*W* pair.	*N*.
+ 7	0	+ 7	+ 0·2917
+ 7	0	+ 7	+ 0·0417
+ 7	0	+ 7	+ 0·0060
− 15	− 6	− 21	− 0·0026
		N_{34}	+ 0·337

Panel 4, 5.

Loads. Left.	Loads. Right.	*W* pair.	*N*.
0	0	0	0
+ 7	0	+ 7	+ 0·0417
+ 7	− 6	+ 1	+ 0·0009
+ 7	+ 6	+ 13	+ 0·0016
		N_{45}	+ 0·044

Panel 5, 6.

Loads. Left.	Loads. Right.	*W* pair.	*N*.
0	0	0	0
0	− 6	− 6	− 0·0357
+ 7	+ 6	+ 13	+ 0·0111
+ 7	0	+ 7	+ 0·0009
		N_{56}	− 0·024

Panel 6, 7.

Loads. Left.	Loads. Right.	*W* pair.	*N*.
0	− 6	− 6	− 0·2500
0	+ 6	+ 6	+ 0·0357
0	0	0	0
+ 7	0	+ 7	+ 0·0009
		N_{67}	− 0·213

TABLE C.—COMPLETE RESULTS.

Point.	N/L.	$F/4$.	G/L.	H/L.	V/L.	Chord force $\times D/L$.	Vertical force.
0	—	—	—	—	+ 3·953	—	− 7·5
	− 0·203	+ 3·75	− 3·953	+ 3·547		7·906	
1	—	—	—	—	+ 4·999	—	+ 3·5
	+ 0·548	+ 2·0	− 1·452	+ 2·548		17·904	
2	—	—	—	—	+ 2·184	—	+ 3·5
	+ 0·614	+ 0·25	+ 0·364	+ 0·864		22·272	
3	—	—	—	—	− 0·973	—	+ 3·5
	+ 0·337	− 1·5	+ 1·837	− 1·163		20·326	
4	—	—	—	—	− 2·707	—	0
	+ 0·044	− 1·5	+ 1·544	− 1·456		14·912	
5	—	—	—	—	− 2·932	—	0
	− 0·024	− 1·5	+ 1·476	− 1·524		9·048	
6	—	—	—	—	− 2·811	—	0
	− 0·213	− 1·5	+ 1·287	− 1·713		3·426	
7	—	—	—	—	− 1·713	—	− 3·0

The third and subsequent columns are completed from the relationships :—

$G/L = N/L - F/4$ . . . . . . from equation (3)
$H/L = N/L + F/4$ . . . . . . „ (4)
$V = H - G$ . . . . „ (5)
(Chord force) $\times D = M - 2N$
Force in vertical $= W/2$.

Simple checks on these results are obtained from the relationship

$2V_n \times$ depth = the difference between the chord forces in panels $(n - 1, n)$ and $(n, n + 1)$,

and by substitution in equation (6) :—

$$V_n - V_{n+1} = 6sN_{n,\,n+1} \,.$$

The former is satisfied to a maximum error of 0·001 and the second to 0·003, which is sufficient agreement, as $6s$ is greater than 5.

Paper No. 5099.

"The Indicator-Diagram and its Interpretation."

By Robert Dowson, B.Sc. (Eng.), M. Inst. C.E.

(Ordered by the Council to be published in abstract form.)[1]

The history of the gradual introduction of the concept of an "adiabatic" expansion or compression is outlined in the Paper, and the inadequacy of the original definition is pointed out. The original definition may be expressed by equating to zero the equation

$$dQ = dE + p \,.\, dv \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

dQ denoting an element of heat communicated to or withdrawn from the working substance, dE the change in internal energy and $p \,.\, dv$ the external work. The equation for an adiabatic change of state is thus

$$dE = -p \,.\, dv = v \,.\, dp, \quad . \quad . \quad . \quad . \quad . \quad (2)$$

leading to $$pv^\gamma = \text{constant.} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

This use of the index γ ($= \frac{K_p}{K_v}$ = ratio of specific heats at constant pressure and constant volume) is permissible only when the adiabatic is "reversible" in the sense in which the term was used by Carnot. It is sometimes found in technical writings that the index γ is used to express an adiabatic change when the process, although "insulated," is subject to internal losses. For a jet of gas flowing through a nozzle, it is correct to argue that, if there is no net gain or loss of energy to the outside during the expansion, therefore the expansion is "adiabatic," but it is not correct on that account to say that the law of expansion is represented by the equation pv^γ = constant. Unless such an expansion is also frictionless, that is, "reversible," it can only be represented by an expression of the form

$$pv^n = k_1 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

which involves very different conclusions as to the change in internal energy and the actual work done.

[1] Type-litho copies of the full Paper can be obtained on loan from the Loan Library of The Institution; a limited number of type-litho copies are also available, for retention by members, on application to the Secretary.

Before dealing with the practical application of this proviso to the reciprocating engine or compressor, it is expedient to consider its application to the steam-turbine and turbo or "flow" type of compressor, because it was in that field that the incorrectness of neglecting it first became apparent. Not much attention was given to the matter during the early development of the theory of the steam-turbine, but later it became necessary, in designing the proportions of steam-turbine nozzles and blades, to formulate some method of taking into account the inefficiency of the latter, which altered the expansion of the steam from the ideal "adiabatic" to something different. To deal with this problem mathematically, a constant "stage" or "hydraulic" efficiency η was assumed for the expansion; equation (2) then became

$$dE = \eta v \,.\, dp. \qquad (5)$$

It was shown by Mr. H. M. Martin that the relation between the index γ in (3) and the index n in (4) was

$$1 - \frac{1}{n} = \eta\left(1 - \frac{1}{\gamma}\right), \qquad (6)$$

and, as a consequence, the available energy in an adiabatic expansion with hydraulic efficiency η, instead of being

$$\int_{p_2}^{p_1} p \,.\, dv = \frac{\gamma}{\gamma - 1}(p_1v_1 - p_2v_2), \qquad (7)$$

was greater, namely

$$\frac{n}{n-1}(p_1v_1 - p_2v_3), \qquad (8)$$

$$= \frac{1}{\eta} \,.\, \frac{\gamma}{\gamma - 1}(p_1v_1 - p_2v_3),$$

the ratio of the two quantities being the reheat-factor R. Here p_1, v_1 denote the initial pressure and specific volume, and p_2, v_2 denote the pressure and specific volume at the end of adiabatic expansion with hydraulic efficiency η = unity; v_3 denotes the final specific volume when $\eta <$ unity.

When the expansion is adiabatic and reversible, the external work done is identical with the available energy, namely

$$\frac{\gamma}{\gamma - 1}(p_1v_1 - p_2v_2),$$

and both are equivalent to the area of the pv-diagram; but with

hydraulic efficiency η, the external work done is less than the area of the pv-diagram and is equal to

$$\int_{p_2}^{p_1} \eta p dv = \left\{\eta \times \frac{1}{\eta} \cdot \frac{\gamma}{\gamma - 1}(p_1 v_1 - p_2 v_3)\right\}$$

$$= \frac{\gamma}{\gamma - 1}\{(p_1 v_1 - p_2 v_2) - p_2(v_3 - v_2)\}, \quad . \quad (9)$$

which is the theoretically-available work minus the losses. It will be observed that the expression for the losses, namely

$$\frac{\gamma}{\gamma - 1}(v_3 - v_2)p_2,$$

is equivalent (in terms of the mechanical equivalent of heat J, the specific heat at constant pressure K_p, and the temperatures T_3 and T_2 corresponding respectively to v_3 and v_2) to $JK_p(T_3 - T_2)$, and shows the dilation of volume from v_2 to v_3 on account of these losses, which leave the turbine as additional waste heat. Hence the area of the pv-diagram for the expansion of steam in a turbine does not represent the external work done unless the expansion, in addition to being " insulated," is frictionless so that η = unity.

The application of this principle to the design of turbo or " flow " compressors is modified by the circumstance that during compression with hydraulic efficiency η, the losses or " extra heat " constitute a dead loss and are not partially recovered as they are during an expansion. The effect of internal losses is thus considerably worse in a compressor than in an expander. Here equation (6) becomes

$$1 - \frac{1}{m} = \frac{1}{\eta}\left(1 - \frac{1}{\gamma}\right),$$

leading to

$$pv^m = k_2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

All this is well known to steam-turbine and turbo-compressor designers, but it does not yet seem to have been applied to the reciprocating steam-engine or air-compressor. The reasons for this seem to be that it is considered, firstly, that such treatment is applicable only to turbine engines, which cannot be directly " indicated " and have only an abstract pv-diagram, and, secondly, that since the reciprocating engine can be directly " indicated," the actual indicator-diagram must represent the work done, provided that the indicator is properly installed. It is, however, on the interpretation of the phrase " properly installed " that the validity of this argument rests.

Much experience has been accumulated and published[1] about the proper method of installing an indicator, but all the writings on the subject neglect a further proviso upon which the truth of the indicator-diagram depends, and that is the "reversibility" of the process which the indicator is to record. It is shown in the Paper that, if the process is not ideal, the same modifications to the actual performance as those outlined above for turbine engines apply to the reciprocating steam-engine and compressor. The area of the actual indicator-diagram does not then truly represent the external work done.

All real working fluids have viscosity, implying internal friction-losses; eddies and turbulence are also generally present, and the action taking place when an imperfect working fluid expands in the cylinder of a reciprocating engine may be illustrated by a mental picture. Imagine a series of fine gauze diaphragms or screens to be inserted in the cylinder as the piston recedes during its out-stroke. The expanding steam has then to filter through these imaginary obstructions as it follows the piston, and the multitude of jets thus formed will dissipate energy as losses. These losses will be returned to the steam as heat, and the pressure-gradient in the cylinder will be somewhat higher throughout the expansion by reason of this heating. It will be evident also that the steam-pressure where the indicator is connected will be somewhat higher than that on the working face of the piston. With the aid of this imaginary process, it becomes possible to appreciate that in a real engine, when internal losses are present, the indicator-diagram over-estimates the external work done. Its area is $\int p.dv. = JRh_a =$ the total available energy, where Jh_a denotes the theoretical available energy and R the reheat-factor due to the internal losses. The external work done by the piston is ηJRh_a, and is equal to the total available energy minus the losses.

In the reciprocating air-compressor, analogous reasoning leads to the conclusion that the pressure on the working face of the piston will be greater than that in the clearance-space to which the indicator is connected: consequently the indicator under-estimates the external work applied. The area of the diagram is equivalent to JRh_a, which now represents the total available energy that is communicated to the working fluid and that would be given out if the fluid were re-expanded adiabatically without internal losses. The work done by the piston, namely $\frac{JRh_a}{\eta}$, is greater than this,

[1] Report on Tabulating the Results of Heat Engine Trials; Appendix VII, also p. 309. The Institution of Civil Engineers, 1927.

because a certain proportion of the energy communicated by it to the working fluid is converted irreversibly into heat. It is possible, therefore, to draw erroneous conclusions from an indicator-card.

The degree of error caused by non-compliance with the condition of "reversibility" of process in actual engines depends upon the magnitude of the internal losses, and in order to deal with this question mathematically, the premise of an hydraulic efficiency η is assumed. The numerical value to be assigned to the coefficient η, and its constancy or inconstancy throughout expansion or compression in a reciprocating engine, are matters requiring the attention of engineers associated with this class of work. If the coefficient η differs appreciably from unity, the effect is sufficiently great to require taking into account (as is well known to designers of steam-turbines and turbo-compressors), and for this reason the mechanical efficiency of a reciprocating engine may be greater than is commonly believed.

It will be seen from the discussion in the Paper that it becomes necessary, in modern thought, to distinguish carefully between ideal or reversible "adiabatic" changes and changes which, although "insulated," are non-reversible. A good definition of the term "adiabatic" has been given by Sir Alfred Ewing,[1] but its significance does not appear generally to have been grasped. It reads as follows:—"Adiabatic expansion or compression means expansion or compression carried out reversibly and without allowing any heat to enter or leave the substance. . . . Adiabatic action would be realized if we had a substance expanding, or being compressed, without change of chemical state, and without any eddying motions, in a cylinder which (along with the piston) was totally impervious to heat." The present Paper has been written to demonstrate the importance of this definition, especially as applied to reciprocating air-compressors.

Throughout the Paper the *pv*-diagram only is used, and no reference to the entropy-diagram is necessary.

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[1] "The Steam Engine and other Heat Engines," 4th edition, p. 57. Cambridge, 1936.

Paper No. 5123.

"The Estimation of Run-off from Areas Subjected to Rainstorms."

By JOHN RUPERT DAYMOND, M.Sc., Assoc. M. Inst. C.E.

(*Ordered by the Council to be published in abstract form.*)[1]

NOTATION.

A denotes the total drainage-area in acres.
$\delta A_{\tau-t}$,, ,, element of area from which the time of flow to the point of outfall from the area is $\tau-t$.
i_t ,, ,, intensity of rainfall in inches per hour at a time t.
I ,, ,, average intensity of rainfall in inches per hour.
q (*with suffix*) denotes the run-off in cusecs at the time given by the suffix.
λ denotes the time of flow from the furthermost point of the drainage-area to the point of outfall from the area.
T ,, ,, duration of a storm in minutes.
ρ ,, ,, coefficient of impermeability of an area.

THE early part of the Paper is devoted to a brief general discussion on existing theory of the relationship between rainfall and run-off. Practically the whole of the theory at the present time is based on a law first proposed by Mr. Emil Kuichling on p. 40 of his Paper on "The Relation between the Rainfall and the Discharge of Sewers in Populous Districts."[2] This law, often misquoted and misinterpreted, states that ". . . in drainage areas *of moderate size*, the heaviest discharge always occurs when the rain lasts long enough *at its maximum intensity* to enable all portions of the area to contribute to the flow. *For large areas, on the other hand, a more elaborate analysis becomes necessary in order to find under what conditions the absolute maximum discharge will occur, . . .*"

In its application to existing methods of estimating run-off, the law is presumed merely to establish a connexion between the time of flow over an area, that is, the time of concentration, λ, and the duration of the storm, T, which will give the maximum rate of flow

[1] Type-litho copies of the full Paper can be obtained on loan from the Loan Library of The Institution; a limited number of type-litho copies are also available, for retention by members, on application to the Secretary.
[2] Trans. Am. Soc. C.E., vol. xx. (1889).

at the point of outfall from the area; no account is taken of the provisos italicized by the present Author in the above quotation. This is an omission of importance, since it is shown in the present Paper that Kuichling's reservation regarding large areas is valid.

Utilizing this law, together with the accepted hyperbolic relationship between I and T,

$$I = \frac{e}{(b + T)^n} \text{ (where } e, b \text{ and } n \text{ are constants)} \quad . \quad . \quad (1)$$

and the "rational" formula,[1] $q = \rho AI$,

$$q_{max} = \frac{Ae}{(b + \lambda)^n} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

is obtained, since it is assumed that the worst conditions prevail when $\lambda = T$.

The Author discusses the justification for and the limitations of formula (2). This formula cannot be applied to large areas, but when utilized for areas of moderate size it is shown by a mathematical analysis that it gives results which agree with Kuichling's conclusions concerning such areas. The formula, however, possesses the disadvantage of not accounting for all the factors which materially influence the run-off. Such factors are :—(*a*) the shape of the area, (*b*) variations in slope of the ground, (*c*) the space-variations in ρ, and (*d*) the shape of the rainstorm curve. Criticism is often levelled against formula (2) because of the absence of a factor to allow for the slope of the area. It is shown that the omission is more apparent than real, since slope is the chief influence upon the value of λ.

The assumption of a constant intensity of rainfall throughout the duration of the storm also invalidates the general use of equation (1), and the equation gives no indication of the behaviour of any one storm, so that the run-off may be seriously under-estimated, since the ratio of i to I may be large.

Arising from G. S. Coleman's[2] "step by step" method of obtaining q from an area subjected to storms of variable intensity, a general theory is proposed, which, while accepting the axiomatic "rational" method, does so for elements of area only. Thus, generally

$$\delta q_T = \delta A_{T-t}\, i_t$$

and

$$q_T = \int \delta A_{T-t}\, i_t \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

[1] D. E. Lloyd-Davies, "The Elimination of Storm-Water from Sewerage Systems." Minutes of Proceedings Inst. C.E., vol. clxiv (1905–6, Part II), p. 41.

[2] "The Estimation of Storm-Water Run-off from Inhabited Areas." Selected Engineering Paper No. 4, Inst. C.E., 1923.

with appropriate limits to the integral, depending upon the relative values of ρ, λ and T.

An independent proof is given for this general type of equation and it is shown how run-off formulas may be easily deduced by representing areas on a "time" rather than on a "length" basis, or, for convenience, by introducing the idea of "isotors"[1] by analogy with contours.

The proposed general theory, from the nature of its derivation, can account for all factors included in existing theory and, in addition, for the items (*a*), (*b*), (*c*) and (*d*), previously mentioned, thus leading to greater exactitude in the estimation of flow.

Regarding i_t as constant in formula (3), it is shown that Kuichling's law is not true for all values of T and λ, but for small values of λ (say from 10 to 20 minutes) the use of formula (2) results in errors that are small compared with the more exact results obtained from formula (3). In addition, an important general conclusion is arrived at, applicable to all areas, namely that, *provided the rain falls at a constant intensity, the storm that gives the maximum run-off is one that lasts for a time less than, or equal to, the time of concentration,* λ, *of the area.* Having regard to the nature of storms which usually cause flooding in inhabited areas (that is, thunder showers of short duration and constant intensity) and to the preceding conclusion, it is suggested that formula (2) is especially suitable for estimating storm flow in sewer-design work.

In its general form, formula (3) is integrable provided that A, $\frac{dA}{dt}$ and $\frac{di}{dt}$ are continuous within the range of integration. Special cases of discontinuities are considered; the case when $\frac{di}{dt}$ is constant ($= m$), but discontinuous, is of especial interest. For, integrating formula (3) by parts, equations of the form

$$q_T = Ai_x + m\int A\,dt \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

are obtained, where x and the limits of integration depend upon the values of τ, λ and T.

Equations of the type given by formula (4) are easily solved by the derivation of a summation-curve of the area. The method is particularly useful, because in practice it may not be possible to express the i_t-curve algebraically, thus eliminating the use of formula (3). Approximating to the rainstorm curve by a series of

[1] An "isotor" may be defined as a line drawn on an area so that the time of flow from any point on the line to the point of outfall from the area is the same.

straight lines, satisfactory results, are, however, obtainable from the arithmetical use of equations represented by formula (4). In order to obviate some of the computation a graphical means is given for the solution of this problem.

Examples are given to show the effect of the shape of the area and of the i_t-curve upon the run-off, and the assumptions made in the development of the general theory are considered, together with the values of q resulting from such assumptions.

ENGINEERING RESEARCH.

THE INSTITUTION RESEARCH COMMITTEE.

Joint Sub-Committee on Special Cements.

At the meeting of the International Sub-Committee on Special Cements held at the Institution in April, the Joint Sub-Committee on Special Cements of The Institution and the British Committee on Large Dams of the World Power Conference presented notes on work which had been carried out at the Building Research Station.

As was explained previously,[1] the Joint Sub-Committee have devoted their attention to an investigation of the heat evolved during hardening and to the solubility of cements. An interim report on solubility was presented in 1936 and was described in the Journal[2]. At the 1937 meeting a note on heat evolution during setting was submitted, and a summary of the results obtained to date is given below.

Other notes were submitted giving pertinent information obtained in the course of allied investigations, including notes on free shrinkage and shrinkage-cracking, and on the testing of Pozzuolanic cements.

Measurements of Free Shrinkage and on Shrinkage-Cracking.

The measurement of the free shrinkage of a cement or concrete under certain arbitrary controlled conditions of test is in itself of little value in deciding on the possibility of such movements causing local failure in a concrete structure. If the tendency to shrink is completely unrestrained, the concrete will attain a new natural configuration without setting up elastic strains, and no trouble will arise. However, in practice there will be some restraint to movement which may be (i) external, due to the monolithic character of the structure or (ii) internal, due to lack of homogeneity of the concrete

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[1] Journal Inst. C.E., vol. 2 (1935–36), p. 175. (February, 1936.)
[2] *Ibid.*, vol. 2 (1935–36), p. 363. (March, 1936.)

or to the presence of reinforcement, or to variability of the magnitude of the shrinkage movement throughout the concrete.

Immediately there is restraint to shrinkage, elastic strains and creep deformations are established, the magnitude of these depending on the degree and manner of restraint and the elastic and creep properties of the concrete.

The shrinkage stress developed is a function of three factors, (i) the free shrinkage, (ii) the modulus of elasticity, and (iii) the ability of the concrete to relieve the induced stresses by creep. Both of these last two factors, and also the tensile strength of the concrete, are functions of the age of the concrete, so that it is to be expected that the possibility of cracking is intimately connected with the rates of shrinkage and hardening.

One example may suffice to show the importance of not relying only on shrinkage measurements when considering cracking. The shrinkages of two concretes, one with ordinary Portland and the other with a rapid-hardening Portland cement, were practically identical. The stresses set up in these concretes when completely restrained were measured and the factor of safety against cracking was determined. The results are given in Table I (p. 274). The concrete made with ordinary Portland cement cracked at an age of 24 days; the creep of the rapid-hardening Portland-cement concrete was smaller than that of the ordinary Portland-cement concrete, particularly at stresses approaching failure, and less relief was therefore obtained, cracking occurring at an age of only 12 days.

In cases where the rate of shrinkage is comparatively slow, it is possible that at the greater ages when failure is approached the effect of creep of the concrete is less important. In such cases the relative shrinkages of various concretes may afford a better guide to the possibility of cracking.

By the courtesy of the International Association for Testing Materials, the notes on the work on heat evolution and on pozzolanic cements are reproduced from the reports which will be published in the Congress Book of the recent Congress of that body held in London.

TABLE I.

Concrete.	Shrinkage $\times 10^{-6}$					Factor of safety against cracking.						
	Age: days.					Age: days.						
	7	14	30	60	90	4	7	10	12	14	21	24
Ordinary Portland cement. 1 : 2 : 4 concrete (by weight). Water/cement ratio=0·60 (by weight) . .	35	60	99	159	186	3·44	1·77	1·31	1·25	1·18	1·07	1·00
Rapid-hardening Portland cement. 1 : 2 : 4 concrete (by weight). Water/cement ratio=0·60 (by weight) . .	35	65	110	160	186	3·34	1·70	1·20	1·00	—	—	—

Comparison of Methods for Measuring the Heat of Hydration of Cements.

By F. M. Lea, D.Sc., F.I.C.

Considerable attention has been paid in recent years to the restriction of the heat of hydration of cements for use in large mass-concrete structures. The methods which have been used for the measurement of the heat of hydration may broadly be grouped into three classes :—

(1) Heat of solution method, as specified in various U.S.A. specifications [1] for low heat cements.

(2) Adiabatic calorimetric methods.

(3) Semi- and non-adiabatic calorimetric methods.

A comparison of the results given by methods (1) and (2) has been made in a report [2] to the Second Congress of the International Commission on Large Dams held at Washington, D.C., U.S.A., in September, 1936, while a method of Class (3), involving the use of thermos flasks, was reported on at the same Congress by the Swedish Committee on Large Dams.[3]

The heat-of-solution method involves the determination of the heat of solution of unhydrated and hydrated neat cement in a nitric-hydrofluoric acid solvent and the calculation of the heat of hydration by difference. The hydrated cement is prepared by curing suitable specimens of gauged neat cement for one day at 22° C. and thereafter at an arbitrarily selected temperature of 38° C. In the adiabatic method the test is carried out on samples of concrete stored under conditions such that no heat is lost from the specimen, the temperature of which rises progressively and proportionately to the amount of heat evolved by the hydrating cement. Two suitable forms of adiabatic calorimeters have been described,[4] one being a simplified apparatus which may be constructed quite cheaply.

A comparison is shown in Table I (p. 276) of the heat of hydration of

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1 For example, Specification No. 566, " Portland Cement for Boulder Dam." U.S. Bureau of Reclamation, Washington, D.C., U.S.A.

2 W. T. Halcrow and F. M. Lea. Question III, " Special Cement."

3 Interim Report, International Sub-committee on Special Cements for Large Dams, p. 46.

4 N. Davey, *Concrete and Constructional Engineering*, vol. 26 (1931), p. 572; Building Research Technical Papers, No. 14 (1933) and 15 (1933) (with E. N. Fox). N. Davey and C. T. Webster, " Simplified Calorimeter." *The Structural Engineer*, vol. 13 (7) (1935), p. 302.

various cements as determined by the heat of solution and adiabatic calorimetric methods. In the latter method the tests were carried out on concrete mixes composed of 1 part cement, 2 parts quartz sand $\frac{3}{16}$ in. (0·476 cm.) down and 4 parts quartz gravel $\frac{3}{8}$ in. (0·935 cm.) to $\frac{3}{16}$ in. (0·476 cm.), proportioned by weight. The water content was 60 per cent. by weight of the cement. The gauged concrete mix was filled into a tin 6 in. (15 cm.) deep by 3 in. (7·5 cm.) diameter, which was closed with a tightly fitting lid and at once placed in the adiabatic calorimeter. By using a suitable regulator the temperature of the calorimeter was caused to rise at the same rate as that of the concrete specimen, from which all heat loss could thus be prevented.[1]

Table I.—Comparison of Heat of Hydration of Cements by Heat-of-Solution and Adiabatic-Calorimetric Methods.

Cement No.	Type.	Heat of hydration: calories per gram of cement.				
		Adiabatic-calorimetric method.			Heat-of-solution method.	
		1 day.	3 days.	7 days.	7 days.	28 days.
1	Portland	50·0	76·6	86·4	95·9	99·7
2	Portland	43·7	63·2	74·3	79·4	94·4
3	Portland	34·8	53·8	66·5	72·0	113·9
4	Portland blast-furnace	24·1	54·0	66·5	68·9	102·4
5	Portland blast-furnace	26·6	49·6	62·3	62·1	87·5
6	Pozzolanic	20·1	34·4	42·4	65·2	79·7

In calculating the heat evolved in calories per gram of cement from the temperature rise observed in the concrete specimens, the specific heat of the concrete has been taken as 0·25, as found by Sheard.[2] For the 1 : 2 : 4 : 0·6 (cement : sand : gravel : water) concrete it follows that a rise in temperature of 1° C. is equivalent to 1·9 calories per gram of cement.

Comparing the heat-of-hydration values at 7 days the heat-of-solution method tends to give somewhat higher values than the adiabatic method for cements of normal heat evolution, but for a cement of low heat evolution, such as No. 6, it gives a much higher

[1] For complete details of the method of preparing the test specimens and carrying out the test, see Interim Report, International Sub-committee on Special Cements for Large Dams, p. 40, Appendix 2 (Tentative Standard Specification for Testing the Heat of Hydration of Cement by an Adiabatic Calorimetric Method).

[2] H. Sheard, "Thermal Constants of Setting Concrete," *Proc. Phys. Soc.*, vol. 48 (1936), p. 498.

value. The difference between the values given by the two methods is to be associated with the difference in the temperature of curing of the test specimens. The rate of hydration, and therefore of heat evolution of cements, increases as the temperature is raised, and the results of heat-of-hydration measurements thus depend on the thermal history of the test specimens. In the heat-of-solution method an arbitrary temperature of 38° C. is selected as the curing temperature subsequent to the first day, whereas in the adiabatic method the initial temperature of the test specimen is 18° C. and its subsequent thermal history is controlled by the rate of heat evolution of the cement under test. With cements of normal heat evolution the temperature of the concrete stored adiabatically rises fairly rapidly so that its average temperature over the first 7 days is not very different from the value of 38° C. used in the heat of solution method, but with cements of low heat evolution the adiabatically stored concrete remains below 38° C. for most of the period. For such cements the heat-of-solution method may be expected to give higher results than the adiabatic method.

In principle the adiabatic method seems preferable to the heat-of-solution method, since in actual large mass-concrete construction the thermal conditions over the early period after the placing of a concrete may closely approach the adiabatic. It has been shown [1] that the temperatures reached in large concrete masses placed at a defined rate can be predicted from measurements of the temperature rise of similar concretes measured in the laboratory by the adiabatic-calorimetric method. The heat-of-solution method probably gives somewhat more closely reproducible results, but the accuracy of the adiabatic method seems adequate for the purpose. A series of comparative tests made at the Building Research Station and in the laboratories of a firm of cement manufacturers by the adiabatic-calorimetric method, using the simplified apparatus, gave the results in Table II (p. 278) :—

The values obtained at 3 days show larger variations between the two laboratories than do those obtained at 7 days.

The adiabatic method enables the complete heat-evolution curve to be determined over the period of the test, but is not very suitable for test periods of 28 days and over. For such ages the heat of solution method is more suitable. The difference between the results of tests by the heat-of-solution and adiabatic methods may be expected to be smaller at ages of 28 days and upwards than at short ages.

The adiabatic method has been tentatively recommended as a

[1] Building Research Technical Paper No. 18 (1935).

standard by the International Sub-committee on Special Cements for Large Dams.

TABLE II.

Cement No.	Heat evolution: calories per gram of cement.					
	3 days.					
	1	2	3	4	5	6
Laboratory 1 . . .	54	63	54	55	34	82
Laboratory 2 . . .	58	70	57	56	32	92
	7 days.					
Laboratory 1 . . .	66	74	67	74	42	98
Laboratory 2 . . .	69	75	71	75	48	102

THE TESTING OF POZZOLANIC CEMENTS.

BY F. M. LEA, D.Sc., F.I.C.

The testing of blended cements containing a mixture of pozzolana with Portland cement raises a number of special problems. In such cements the Portland cement fraction hydrates and develops strength at the usual rates characteristic of this material, but the reaction of the pozzolana is a slower and more gradual process. The strength developed at ages of 7, and even 28 days, by pozzolanic cements depends primarily on the amount and rate of hardening of the Portland cement present, and only secondarily on the pozzolana. The ultimate properties, such as the strength at long ages and the resistance to attack by sulphate-bearing waters, however, depend very markedly on the pozzolana.

As alternative methods for testing pozzolanic cements, one may either test the blended cement or require separate tests on the pozzolana and Portland cement constituents. The former method is adopted in certain national specifications which cover pozzolanic cements as well as Portland and other types. Though the application of the same strength tests as are used for Portland cement will, provided the test mortar is of plastic and not dry consistence, afford a check on the rate of strength development at early ages of a pozzolanic cement, it affords no test of the quality of the pozzolana present. The method of independent testing of the pozzolana and

Portland cement constituents has been adopted in the specification [1] for pozzolanic cement for Bonneville dam on the Columbia river, U.S.A., and is also to be found in the German specification for trass. In both these cases the pozzolana is tested in a lime mortar. Although lime mortar tests are of value when considering lime-pozzolana mixes they do not appear to afford a satisfactory indication of the value of a pozzolana for use in mixtures with Portland cement.

Specification tests for pozzolanic cements should afford a measure of the strength which may be expected at early ages and also of the quality of the pozzolana present. Investigations on pozzolanic cements have been in progress at the Building Research Station

TABLE I.—COMPARISON OF STRENGTH TESTS ON POZZOLANIC CEMENTS.

Strengths of cements composed of 60 per cent. Portland cement, 40 per cent. pozzolana by weight expressed as a percentage of the strength of the unsubstituted Portland cement in the same test and at the same age. All test specimens stored in water at 18° C.

		Pozzolana.			
		None.	A	B	C
Tensile strength 1 : 3 dry consistence mortars (8 % water).	3 days	100	75	86	89
	7 ,,	100	81	89	93
	28 ,,	100	95	104	104
Tensile strength 1 : 3 plastic [2] consistence mortars.	7 ,,	100	51	59	59
	28 ,,	100	61	66	74
Compressive strength 1 : 2 : 4 concrete. Water content 60 % by weight of cement.	7 days	100	57	59	55
	28 ,,	100	60	67	60
	6 months	100	67	75	84
	1 year	100	73	94	94

over the last 8 years. In the more recent work particular attention has been paid to methods of test suitable for use in specification.

The substitution of pozzolana for Portland cement reduces the strength of concrete at early ages but, as has been shown by much previous work, no corresponding reduction is found in the strength of standard mortars of dry consistence. Tests on such mortars are therefore misleading. Mortars of plastic consistence, corresponding to water contents of 11 per cent. or over on 1 : 3 cement standard sand mortars, offer a much improved indication of the strength of concrete at similar ages. This is illustrated by the data given in

[1] J. L. Savage (U.S. Department of the Interior), "Special Cement for Mass Concrete," p. 161. Report presented at the Second Congress of the International Commission on Large Dams, Washington, D.C., 1936.

[2] Consistence as determined by Vicat plunger test with 500 grams additional load.

Table 1 (p. 279) for the strengths obtained with pozzolanic cements prepared by mixing 40 per cent. pozzolana with 60 per cent. of the same Portland cement throughout. The strengths in all cases are shown as a percentage of that of the corresponding specimens at the same age made with a cement composed of 100 per cent. of the same Portland cement. Pozzolana A was a material of poor quality and pozzolanas B and C, materials of good quality, as shown by long-age strength and sulphate-resistance tests.

The water required for a mortar of standard plastic consistence may be conveniently determined by a method similar to that described by Feret at the Brussels meeting in 1906 of the International Association for Testing Materials. The test is carried out on the mortar using a Vicat plunger and mould as described in the British Standard Specification for Portland Cement (No. 12, 1931), but adding an additional load of 500 or 2,000 grams to the plunger and determining the water content required for the plunger to penetrate to a distance of 10 mm. from the base of the mould. Comparative tests in three different laboratories have shown that the method of mixing the mortar and filling the mould can be sufficiently closely defined to give adequately reproducible results. For Portland cement the water content obtained corresponds to about 12·5 per cent. by weight of the dry mortar when the 500 grams additional load is used, and 11·0 per cent. with the 2,000 grams additional load. With different pozzolanic cements the values vary from a little below these figures to about 2 per cent. higher.

Compression tests on pozzolana mortars of dry consistence, though less misleading than tensile tests, still appear unsatisfactory. Compressive tests on plastic mortars may eventually be preferred to tensile tests, but adequate data are not yet available.

The strength tests at short ages do not differentiate between pozzolanic cements containing good and poor pozzolanas, since the strength obtained may be varied by changes in the proportion and rate of hardening of the Portland cement present. Additional tests are therefore required. Investigations now in progress indicate that a satisfactory test of the quality of the pozzolana present may be obtained from the strengths developed in two sets of plastic mortar briquettes, one cured for one day in moist air at 18° C. and two days in water at 18° C., and the other cured one day in moist air at 18° C., followed by 24 hr. in water at 50° C., and then one day in water at 18° C. The ratio of the difference between the strengths developed under these two conditions to the sum of the strengths (i.e. (50° strength — 18° strength)/(50° strength + 18° strength)) has been found to show a fair correlation with the resistance of pozzolanic cement mortars to attack by magnesium and sodium sulphate

solutions and to the strength increase at long ages. Pozzolanic cements containing poor pozzolanas, or too low a proportion of a good pozzolana to have a high resistance to sulphate attack, and also Portland cements, give values for this ratio below 0·25 to 0·30, whereas satisfactory pozzolanic cements show values above these figures. Another test which has been found of some value is a form of lime extraction test on the set neat cement. By a combination of minimum strength requirements at 7 and 28 days with the tests discussed above, and with the inclusion of the normal requirements for setting time, soundness, etc., a reliable form of specification for pozzolanic cements seems feasible.

A complete report of the investigations carried out at the Building Research Station on pozzolanic cements is in course of preparation. Brief reports on the work have been published in the Annual Reports of the Building Research Board for 1934 and 1935.

RESEARCH WORK IN ENGINEERING AT QUEEN MARY COLLEGE (UNIVERSITY OF LONDON), MAY 1937.

The following is a brief description of recent research work carried out in the Departments of Civil and Mechanical Engineering and of Electrical Engineering.

Department of Civil and Mechanical Engineering.

On the civil engineering side extensive model-experiments on foundation-pressures in sand and clay have been carried out for the British Electrical and allied Industries Research Association. In connexion with the erection of pylons, methods have been developed of estimating the holding power of poles erected in various soils from observation of the behaviour of model poles erected on the site. A more fundamental research into the cohesion and shear resistance of clays has also been carried out. The distribution of vertical pressure under a foundation has been studied by preparing a model in which shot is embedded in a rubber continuum and then measuring the indentations produced by the shot in a horizontal lead sheet. Arrangements have been made, in co-operation with

the Building Research Station, for an investigation of the settlement of the foundations of a new block of buildings about to be erected on the College site.

In connexion with mechanical engineering, research has been conducted on diesel engines into the temperatures and thermal stresses set up in internal-combustion engines. In addition, the temperatures of the working fluid have been investigated. In the solid-injection engine a study has been made of flame-propagation and combustion. There is now under consideration a research into the initiation of the flame in the internal-combustion engine. In connexion with the ammonia refrigerator, a study has been made of mass-flow and its measurement with a view to improving its performance.

On the subject of materials, a research has been directed to the making of accurate comparisons of the elastic constants of specimens cut from rolled steel along and transverse to the direction of rolling.

Since 1935 research work in mechanical engineering has been drastically curtailed owing to building operations and re-arrangements, and to the replacing of obsolete plant by more modern equipment.

There have in addition been carried out certain researches of a confidential nature.

Department of Electrical Engineering.

A research is being directed towards the development of a rapid method for the accurate determination of resistance/temperature coefficients which will be suitable as a works method. A satisfactory apparatus has been produced and measurements on a series of materials are in progress. A detailed investigation has been made of phenomena associated with the rubbing contact of the brushes of electrical machines.

A large proportion of the research is in connexion with high frequencies. A thermal method, suitable for industrial use, of measuring dielectric losses at radio frequencies is being developed in conjunction with the Electrical Research Association. Provisional results have already been obtained and are being put into practice by interested firms. Study is being made of the behaviour of selenium rectifier cells. Methods are being devised for the measurement of interference produced in wireless receivers by insulators under various conditions. A research into television has been initiated, and apparatus is being developed and calibrated for the measurement of strength of signal received from the B.B.C. An investigation of the properties of "Ferrocart" has just been completed.

A new high-voltage laboratory was opened in May, 1936. This contains apparatus able to produce voltages of the order of 500,000 at ordinary power frequencies (50 cycles), and of the order of 1,000,000 as an impulse voltage. Researches are being carried out into phenomena associated with these high voltages.

The foregoing researches are being carried out under the direction of Professor E. H. Lamb, D.S.C., D.Sc., Professor of Civil and Mechanical Engineering and Dean of the Faculty, and Professor J. T. McGregor-Morris, Professor of Electrical Engineering.

REPORT OF THE NATIONAL PHYSICAL LABORATORY FOR THE YEAR 1936.[1]

The Report of the National Physical Laboratory for the year 1936 shows a general increase of activity in the various departments, and it is satisfactory to note that industry appears to be taking increasing advantage of the facilities for testing and research offered by the Laboratory. This is particularly evidenced in the work of the William Froude laboratory, where the designs of nearly 80 per cent. of the merchant ships under construction in Great Britain during 1936 were based upon the Laboratory tests.

Probably the highest precision yet obtained in any physical measurement, namely a few parts in one hundred thousand million, was attained in an investigation into the effect of the earth's motion through space on the vibrations of a rod according to its orientation. A negative result was obtained which is in accordance with the theory of relativity. A further work of great precision is a determination of the value of the International Ohm. It has been found to equal 1·00050 ohms in absolute units. A common unit of measurement has been evolved correlating X-ray and gamma-ray intensity. Of importance in connexion with the investigation into noise from mechanically-propelled vehicles which is being carried out for the Ministry of Transport is the development of an objective noise-meter. Further progress has been made in the study of the structure of materials by X-ray and electron-diffraction methods.

An investigation has been made of the characteristics of various

[1] Published by H.M. Stationery Office.

forms of conductors for A.C. transmission. The protection of transmission networks against damage and consequent interruption of supply has been studied on behalf of the Central Electricity Board. In the Photometry laboratory illumination requirements and glare have been investigated.

Radio research has included fundamental investigations into the propagation of waves and the solution of problems which have arisen in practice. In addition a research into radio aids to navigation has been carried out for the Air Ministry.

In the Metrology department work has been directed towards the direct measurement of end gauges in terms of the wave-length of light. Connected with this is a redetermination of the refractive index of air, an exact knowledge of which is essential to determine the wave-length in vacuo. Work is in progress as a result of which it is hoped to measure the value of gravity at the Laboratory to an accuracy of one part in a million.

Engineering researches commenced during the year have included work designed to lead to the rational design of high-duty crank shafts for aeroplane engines and the development of a high-speed wind-tunnel. A definite stage has been reached in the long-range research into the fundamental aspects of the deformation and fracture of metals. From work on specimens consisting of several large crystals it has been shown that the influence of the boundaries on slip-band distribution is slight. Whatever the nature of the stressing, the deformation of the crystalline structure is identical, and a common condition represents the fracture stage, failure occurring by progressive breakdown of the grain to a limiting size of crystallite. A comprehensive research on corrosion-fatigue has been concluded. It has been shown that air may act as a corrosive medium, oxygen in the presence of water vapour being the active agent. Coatings affording almost complete protection against corrosion-fatigue, which may be applied to steel, have been found. Interim and subsidiary reports have been issued in connexion with the researches on wear of metals, on the behaviour of metals at high temperatures, and on pipe flanges and bolted connexions.

A statistical examination has been made of wind-pressure effects in a large built-up area. Experiments have been concluded on the Severn bridge giving information relating to a wind front of ½ mile. In lubrication research the presence of small quantities of water in the lubricant was found to have an appreciable effect on the seizing temperature and friction. Other researches in the Engineering department include efficiency tests on driving belts, wheel impact with self-propelled vehicles, construction in thin sheet metal in which the failure of panels stiffened in various manners and subjected

to edge loading is studied, a statistical examination of the strength of welded joints, and a study of the stresses in bridge members due to known wheel-impact forces.

In Metallurgical research a study of the constitution of alloys of iron has been continued. A large number of investigations of materials which have failed in service have been made. Accelerated corrosion tests have been carried out on various coatings, metallic and non-metallic. An important part of the work is the study of metals in the highest state of purity. Research has continued into light alloys of magnesium and of aluminium and into materials at high temperatures.

One of the most important researches of the Aerodynamics department concerns the drag of wings and bodies at high speeds. Present-day speeds are rapidly approaching the value at which the compressibility of the air can no longer be neglected. At high speeds the drag due to projecting rivet heads is found to be of decreasing importance. A comparison has been attempted of the performances of an aeroplane, an autogiro and a helicopter of the same power. An important side of the work concerns stability and control in flight. Advances have been made in the study of fluid flow and the nature of turbulence. Theory based on statistical considerations of spatial and time variations of velocity has received experimental support. Research has also been carried out on air-screws and on the flutter of wings.

In the William Froude laboratory the influence of waves on the resistance, propulsion and pitching of ships has been studied. Research has also been directed to the optimum design of screw propellers, the channels of cavitations, and the reduction of vibration of ships.

REPORT OF THE ROAD RESEARCH BOARD FOR THE YEAR ENDED 31st MARCH, 1936.[1]

The suitability of a subsoil as a road foundation is one of the subjects considered during the year. Since deformation depends upon capillary forces as well as on loading and impact, it is not possible to estimate supporting power directly by simple loading

[1] Published by H.M. Stationery Office.

tests, and work has been directed towards the development of methods of soil sampling and a fundamental study of the characteristic properties of soils.

It has long been maintained that flaky and elongated material is undesirable in road aggregates and a practical method for the assessment of flakiness has been devised.[1] In order that a higher quality of concrete may be maintained it is necessary to keep a careful control over water-content and a machine has been devised whereby, by means of vibrations, the sand and stone are brought to a constant water-content before mixing into concrete.[2] A study has been made of the effect of the design of plain and reinforced-concrete slabs on cracking, curling and the like.

A large amount of research is being carried out on bituminous carpets. Failure may occur as the result of disintegration, irregular wear or deformation of the wearing coat. The length of time required and the variability of conditions render it difficult to accumulate definite information from records of actual road-wear. Accelerated large-scale tests, supplemented by small-scale laboratory tests, have therefore been carried out. The large-scale tests have been carried out on circular tracks with three specially constructed road-machines, the largest of which, 110 feet in diameter, has been completed during the year. It is hoped to investigate surface dressings, thin surfacing coats and road behaviour generally.

Another form of failure is by the formation of a slippery surface. A method of assessing the frictional properties of a road surface by means of a special motor-cycle and side-car combination has been developed [3] and much valuable information has been accumulated. A theory has been advanced to explain the effects which have been found, but further study is necessary before the causes of skidding can be definitely established. The information obtained with the above-mentioned apparatus refers of necessity to tires with smooth treads and lateral slipping, and in this respect differs from actual conditions. A special apparatus has been designed to investigate

[1] "The Shape of Road Aggregate and its Measurement." Road Research Bulletin No. 2.

[2] "The Control of the Moisture Content of Aggregates for Concrete, introducing a new Vibration Method." Road Research Technical Paper No. 4.

[3] J. Bradley and R. F. Allen, "Factors affecting the Behaviour of Rubber-tyred Wheels on Road Surfaces." Proc. Inst. A.E., vol. xxv (1930–31), p. 63.

R. G. C. Batson, G. Bird and R. E. Stradling, "Road Engineering Problems: Judging the Slippery Road." Journal Inst. C.E., vol. 2 (1935–36), p. 443. (April, 1936.)

G. Bird and W. J. O. Scott, "Measurement of the Non-skid Properties of Road Surfaces." Road Research Bulletin No. 1.

the effect of tire diameter and load on the tendency to slip. The effects, however, are found to be small.

Other apparatus has been devised for the measurement and recording of surface irregularities. At present very little is known of the effects of transient loads on bituminous road surfacing. The material will behave to some extent as an elastic solid, but as waving of the surface does occur in practice, creep must under certain conditions take place. In some cases there may be an effect akin to "fatigue" of metals in which case failure would probably occur as a result of distintegration. Work is in hand to attempt the correlation between measurements of surface irregularities and direct impact measurements. Although the work is still in its early stages the knowledge already gained has enabled useful comparisons of various types of road to be made.

NOTES ON RESEARCH PUBLICATIONS.

Measurements and Measuring and Recording Instruments.

The Report of the 26th National Conference on Weights and Measures has been published. *U.S. Nat. Bur. Stand. Misc. Pubn. M. 157.*

In *Travaux de L'Association Internationale de Géodésie*, Vol. 13, are given reports of progress from different countries submitted at the 6th General Assembly held in Edinburgh, Sept. 1936.

Research into the performance characteristics of a water-current meter in water and in air, *U.S. Nat. Bur. Stand. J. Research*, **18,** *351*, shows that as regards the Price type meter no appreciable errors occur in practice as a result of density effects.

A method for determining the modulus of elasticity by flexural vibration which is particularly applicable to building materials such as tiles, slates, and bricks is described in *Phil. Mag., 1937*, **23,** *96*, and on *p. 361* a dynamical method for the measurement of Young's modulus for imperfectly elastic metals and the application of the method to nickel and some of its alloys is given.

Engineering Materials: Properties and Testing.

The Building Research and Public Works Laboratories of Paris, their Objects, and their Methods were described in a Paper read before the *Soc. Ingénieurs Civils de France* (*British Section*) on the 20th January, 1937. *U.S. Bureau of Mines Bull., 393*, contains contributions to the data on theoretical metallurgy: *No. 5* of the series deals with heats of fusion of inorganic substances and *Bull. 394, No. 6*, gives a revision of the entropies of inorganic substances. A theory of fracture of brittle materials is described in *Zeitschrift für Physik*, **103,** *495*, and in *J. Inst. Metals*, **4**, (3) *Metallurgical Abstracts*, *79*. The effect of pressure on the modulus of rigidity of several metals and glasses is dealt with in *Physics*, **8,** *129*.

Timber.

Part 3 of the series on strength tests of structural timbers, *Department of Scientific and Industrial Research, Forest Products Research*

The figure in heavy type is the number of the Volume; that in brackets the number of the Part; and that in italic type the number of the Page; in references to "Engineering Abstracts," the number of the Abstract is given.

Records, No. 15, dealing with the development of safe loads and stresses, with data on Baltic Redwood and Eastern Canadian spruce, has been published.

Cement and Concrete.

The influence of grain size and of various salts upon Portland cement is discussed in *J. of Applied Chemistry* (*U.S.S.R.*), *1936*, **9**, *1937–50* (*In English on p. 1950*). Various methods of defining numerically with particle-size and shape are compared in *Chem. and Ind.*, **56**, *149*. Studies on the relation between characteristics of blast-furnace slag and other coarse aggregates, and the properties of resultant concretes, are given in *Am. Soc. Testing Materials, Proc.*, **36**, *297*. The influence of temperature upon the hardening of cement concrete is discussed in *Gén. Civ.*, **110**, *229* (*Eng. Abs.*, **74**, 34), and the influence of temperature on the hardening of aluminous cement in *Tsement 5* (*2*), **33** (*Eng. Abs.*, **74**, 33). The results of research on the effect of temperature on the stress deformation of concrete are given in *U.S. Nat. Bur. Stand. J. Research*, **18**, *195* (*Eng. Abs.*, **74**, 14). As a result of observed failure, tests were carried out on the volume constancy of cinder-concrete and are described in *Bautenschutz*, **7** (*8*) *89*. The results of tests of pressure storage and permeability, etc., of oil-well cements are given in *Rock. Prod.*, **38** (*12*) *54*. Sodium and magnesium soundness tests are discussed in *Am. Soc. Testing Materials Proc.*, **6**, *327*. An article on Silt Asphalt Cement in Concrete is contained in *Beton und Eisen*, **36**, *86* (*Eng. Abs.*, **74**, 32).

Metals.

Modulus of Elasticity and Damping in relation to the state of the material is considered in *Zeit. Metall.*, **29**, *116* (*Eng. Abs.*, **74**, 23). Calculations regarding the structure of polished metal surfaces are drawn in *Phil. Mag.*, **23**, *397*. Experimental investigations of the distribution of grains by fatigue and static stressing by means of X-ray examination indicate considerable plastic flow under fatigue stresses in the safe range of stress, *Metals and Alloys*, **8** (*1*) *13*. The influence of mechanical vibrations on the tensile properties of constructional materials is considered in *Zeitschrift Metall.*, **29** (*2*) *60* (*Eng. Abs.*, **74**, 20). Extended-time creep tests at 1,000° F. on carbon steel are described in *Am. Soc. Testing Materials Proc.*, **36**, *139*, and on *p. 118* is an article on damage and overstress in the fatigue of ferrous metals. The influence of inherent heat stresses upon the fatigue strength of steel is dealt with in *Ver. deu. Ing.*, **81**, *276* (*Eng. Abs.*, **74**, 41). A report on the Corrosion Conference, 1935, has been published, Berlin, 1936 (*Verein Deutscher Ingenieure*

Verlag). A method of assessing the productive value of zinc coatings on iron and steel otherwise than by accelerated corrosion tests is discussed in *J. Inst. Metals*, **61,** *225*. A quick method of testing corrosion is given in *Tek. Tids.* (*Kemi*), **67,** *9*. In a series of soil corrosion studies, 1934, field tests of non-bituminous coatings for underground use are described in *U.S. Nat. Bur. Stand. J. Research*, **18,** *361*. Creep tests on lead and lead alloys extending over 3 years are described in *Am. Soc. Testing Materials Proc.*, **36,** *170*.

Other Materials.

Research at Leeds University on transverse elasticity of natural stones is described in *Proc. Leeds Phil. and Literary Soc.*, **3** (*5*). Tests of heat resisting materials at high temperatures are given in *Kaiser Wilhelm Inst. Eisenforsch*, **18,** *247* (*Eng. Abs.*, **74,** 29).

Engineering Materials: Production, Manufacture, and Preservation.

The factors influencing the selection of a preservative treatment for wood against rot and insect-attack are considered in *Bautenschutz*, **7** (*12*) *140*. The grading of aggregates for concrete is discussed in *Chemistry and Industry*, **56,** *296*. The control of the quality of concrete by means of its density is suggested in *Schw. Bauzeitung.*, **109,** *137* (*Eng. Abs.*, **74,** 35). A method of rating the characteristics of fresh concrete by visual observation of various factors is given in *Am. Soc. Testing Materials, Proc.*, **36,** *372*. In *Chemistry and Industry*, **56,** *361*, the preparation of metal surfaces for painting is discussed.

Structures.

Mass Structures.

The bearing power of soils is discussed, the results of loading tests are given, and a drop penetration test which has been evolved and its behaviour with various soils is described, in *J. Franklin Inst.*, **223,** *443*. The present state of foundation materials research in Germany is summarized in *Beton und Eisen*, **36,** *77* (*Eng. Abs.*, **74,** 50). The lateral stability of the counterforts of large multiple-arch dams is discussed in *Science et Industrie*, **21,** *69* (*Eng. Abs.*, **74,** 53). Measurements of the movement of boulder dam due to grouting are detailed in *Civ. Eng.*, **7,** *282* (*Eng. Abs.*, **74,** 56). A method for the predetermination of the rise of temperature in poured concrete dams is given in *Zement*, **25,** *743*. The stresses in subaqueous tunnel tubes

of concrete with flexible horizontal ties are analysed in *Am. Soc. Civ. Eng. Proc.*, **62,** *1519.*

Framed Structures.

A Paper on eccentrically-loaded columns is contained in *Tôhoku Imp. Univ.*, **12** (*2*) *23* (*Eng. Abs.*, **74,** 69). A mathematical solution of the symmetrical flexure of an angle-iron is given in *Phil. Mag.*, **23,** *745.* A method of stress analysis for Vierendeel trusses is explained in *Am. Soc. Civ. Eng. Proc.*, **63,** *216* and *335.* Experiments on welded plate girders are described in *Bautech*, **15** (*Stahlbau*, **10**), *33* (*Eng. Abs.*, **74,** 64). The influence of the length of rivet upon the inherent stresses in unloaded riveted joints is considered in *Wärme*, **60,** *175* (*Eng. Abs.*, **74,** 68). Comments on the new Austrian regulations for reinforced concrete are made in *Concrete and Constructional Engineering*, **31,** *653.* The principles of earthquakes and reinforced concrete are discussed in *J. Am. Concr. Inst.*, **8,** *223.*

Transformation, Transmission and Distribution of Energy.

The influence of the water in boilers on the formation of fractures due to fatigue is discussed in *Science et Industrie* (*Mécanique*), **21,** *12* (*Eng. Abs.*, **74**, 109). In *J. Inst. Aut. Engineers*, **5,** (7), *25*, is given the report of the Research and Standardization Committee on Valve Seat Wear.

The synchronization of induction motors without mechanical connection is discussed in *Elec. World*, **107,** *720* (*Eng. Abs.*, **74,** 128). The properties of a dielectric containing semi-conducting particles of various shapes are considered in *J. Inst. Elec. Engineers*, **80,** *378.* A lecture on deterioration in paper-insulated cables is published in *J. Jun. Inst. Engineers*, **47,** *283.* Damage to communication lines as a result of lightning is considered in *Elek. Zeit.*, **58,** *337* (*Eng. Abs.*, **74,** 137). The transmission of alternating-current power with small eddy-current losses is discussed in *J. Inst. Elec. Engineers*, **80,** *395.* The 9th Annual Report of the Central Electricity Board and the Report of the Electricity Commissioners for the year 1936 have been published.

Mechanical Processes, Appliances, and Apparatus.

The following articles on welding have been noted: Radiographic examination of welds, *Welding Industry*, **5** (*3*) *104*; the Second Report of the Welding Research Committee, *Proc. Inst. Mech. Engineers*, *1936*, **133,** *5*; and in the same Proceedings, *p. 295*, a Paper on residual stresses in arc-welded plates.

Experiments on lubricating oil in vehicle engines are described in *Ver. deu. Ing.*, **81,** *347* (*Eng. Abs.*, **74,** 163). In *Technical Publications of the International Tin Research and Development Council Series A., No. 51*, Part III of the research on thin layers of tin and other metals deals with the interaction between metals and lubricating oils.

Specialized Engineering Practice.

Transport.

An investigation of the consolidation of fillings, describing an examination of 100-year-old road embankments is given in *Strasse*, **3,** *520.* A further article describing a method of consolidation appears on *p. 562* of the same journal; and dynamic investigations on embankments are dealt with on *p. 521.* Petrology and modern road problems: a summary of the properties required of stone for use in setts, macadam, chippings, and concrete aggregate, is given in *Science Progress*, **31,** *425.* The effect of the quality of reinforcing steel in concrete slabs is discussed in *J. Am. Concr. Inst.*, **8** (*1*) *1.* A method of determining the vertical elastic stressing of a road by lorries from the measured vibrations is given in *Strassenbau*, **27** (*7*) *110.* A Progress Report on the anti-skid properties of road surfaces; research carried out for the Highway Research Board, is given in *Highway Research Abs.* (*Washington, D.C.*), *1937* (*36*) *5.* Bitumen, its sources, development, and use on roads is discussed in *Science Progress*, **31,** *666.* Information regarding the fatigue strength of vehicles for rough terrain is given in *Am. Weld. Soc.*, **16** (*1*) *Welding Res. Supp.*, *1.* The Third Interim Report to the Ministry of Transport of the Department of Scientific and Industrial Research Committee on Noise in the operation of mechanically-propelled vehicles has been published.

In *Proc. Am. Soc. Civ. Eng.*, **63,** *41*, is a symposium on the national aspects of flood control, and on *p. 539* is given the report of the Committee on Floods. Model experiments on wave-breaks are described in *Ann. Ponts et Chaussées*, **106**-*ii*, *733* (*Eng. Abs.*, **74,** *171*). The effect of cutoffs in lowering the flood-crest on the Mississippi is discussed in *Eng. News-Record*, **118,** *265* (*Eng. Abs.*, **74,** 175). The following Aeronautical Research Committee Reports and Memoranda have been noted: *No. 1727*, The effect of blade twist on the characteristics of the C.30 Autogiro; *No. 1729*, Wind-tunnel tests of high-pitch airscrews. Part II, Variations of blade width and blade section; *No. 1740*, Note on performance data for honeycomb radiators in a duct; *No. 1744*, On the erosion of sparking-plug electrode materials and the variation of sparking-plug voltage;

No. 1742, The wing stiffness of monoplanes ; *No. 1748,* The stressing of a particular rigid-jointed fuselage under bending loads ; *No. 1754,* Turbulent flow in a circular pipe.

In *J. Roy. Aer. Soc.,* **41,** the following articles have appeared : *p. 257,* Recent progress in the design of civil flying boats ; *p. 284,* Aerodrome design ; *p. 306,* Notes on the design of aeroplanes for attaining high altitudes ; *p. 322,* The part played by skin friction in aeronautics. The following Reports of the U.S. National Advisory Committee for Aeronautics have been noted : *No. 579,* A study of the two-control operation of an airplane ; *No. 582,* Primary failure of straight centrally-loaded columns (*Eng. Abs.,* **74,** 70) ; and *No. 584,* Strength of welded aircraft joints. The following articles have appeared in *Luftfahrt.* **14** ; *p. 93,* Theory and tests of the strength of shell bodies (*Eng. Abs.,* **74,** 84) ; *p. 116,* Influence of rivet spacing upon the compressive strength of stiffened duralumin shells (*Eng. Abs.,* **74,** 85) ; *p. 121,* The stress-bearing width of plates in compression (*Eng. Abs.,* **74,** *67*) ; and *p. 137,* Stability of orthotropic cylindrical shells of elliptic cross-section in pure bending (*Eng. Abs.,* **74,** 87).

Water-Supply and Sewage-Disposal.

Experiments on the corrosion of galvanized hot-water tanks for household use with various types of water are described in *Water Works and Sewerage,* **83,** *384.* The protection of the intake grids of water-turbines against ice is discussed in *Gidrotechnicheskoe Stroitelstvo, 1937* (*3*) *28* (*Eng. Abs.,* **74,** 93).

The following articles in *Sewage Works J.,* **9,** have been noted : *p. 6,* Elutriation of digested sludge (*Eng. Abs.,* **74,** 196) ; *p. 41,* Comparison of compressed-air and mechanical aeration of activated sludge. (*Eng. Abs.,* **74,** *194*) ; *p. 63,* Chlorination of activated sludge (*Eng. Abs.,* **74,** 193) ; *p. 34,* Influence of phosphorus and nitrogen on biochemical oxygen demand (*Eng. Abs.,* **74,** 192).

Lighting, Heating, and Acoustics.

A Paper describing work at the National Physical Laboratory on the thermal conductivity of insulating materials and methods of testing the same is given in *J. Inst. Heating and Vent. Engineers,* **5** (*49*) *21.* The correlation of existing knowledge regarding the selection of suitable heat insulating materials for various purposes appears in *J. Inst. Fuel,* **10,** *223.* Heat-transfer and pressure-drop in rectangular air-passages is discussed in *Industrial and Engineering Chemistry,* **29,** *337* (*Eng. Abs.,* **74,** 104).

The effect of air coupling in acoustic insulation by means of

elastic supports is discussed in *Phil. Mag.*, **23,** *154,* and in the same volume, *p. 161,* is a Paper on the sound insulation of single and complex partitions.

Telegraphy and Telephony.

The design and manufacture of modern receiving valves is described in *J. Inst. Elec. Engineers*, **80,** *401,* and on *p. 440* is an article on the measurement of the high-frequency resistance of single-layer solenoids.

Miscellaneous.

The Japanese Journal of Engineering, 1937, **15,** contains abstracts of Japanese papers in various branches of engineering published in 1935. In *Proc. Roy. Soc. Series A*, **159,** *473,* is a Paper on the similarity theory of turbulence and flow between parallel planes and through pipes; on *p. 496* a Paper on flow in pipes and between parallel planes; and on *p. 592* the flow under gravity of an incompressible and inviscid fluid through a constriction in a horizontal channel. The resistance to the flow of water along smooth rectangular passages and the effect of a slight convergence or divergence of the boundaries is discussed in *Phil. Mag.*, **23,** *490.* Boundary-layer instability in diverging channels is dealt with in *Zeit. ang Math.*, **17,** *8* (*Eng. Abs.*, **74,** *189*). The 22nd Report of the Department of Scientific and Industrial Research Committee on the investigation of atmospheric pollution, giving observations in the year ended March, 1936, has been published.

NOTE.

OBITUARY.

ELIHU THOMSON was born at Manchester on the 29th March, 1853, and died at Swampscott, near Lynn, Massachusetts, on the 13th March, 1937. At the age of five he left with his parents for Philadelphia in the United States and was educated in the public schools of that city, graduating with honours from the Central High School in 1870. After serving 6 months as analyst in a Philadelphia laboratory, he was appointed in the autumn of that year Assistant Professor of Chemistry and Physics in the Central High School, a position which he held until he was promoted to the Chair of Chemistry and Mechanics in 1876.

In 1880 he accepted the position of Electrician and Chief Engineer to the American Electric Company, the name of which was changed to the Thomson-Houston Electric Company. He took charge of the commercial development of the Thomson-Houston arc-lighting system, based on his patents some of which were taken out jointly with his former colleague at the Central High School, Professor Edward J. Houston. He was largely responsible for the great expansion of this Company, and many of the fundamentally important inventions upon which the business was based were due to him.

In 1892, the Company was merged with the Edison General Electric Company to form the General Electric Company. Professor Thomson continued to act in a consultative capacity, introducing many valuable improvements in the practical use of electricity for lighting, for railways and for power-transmission. He was responsible for a great number of inventions which advanced the progress of electrical engineering, particularly in connexion with the generation, use and control of alternating currents. By his pioneer discoveries and inventions in connexion with arc-lighting, the art of electric welding by the resistance method, the perfection of indicating instruments and the proper use of lightning arrestors of all forms, his leading position as one of the foremost engineers in the United States has been recognized all over the world.

Professor Thomson was elected a Member of The Institution in 1895 and in 1924 delivered the James Forrest Lecture, taking as his subject "The Unsolved Problems of Electrical Engineering." He was a Fellow of the American Institute of Electrical Engineers, an Honorary Member of the Institution of Electrical Engineers (London), a Member of the Academy of Arts and Sciences, and of the

National Academy of Sciences of the United States, and also Honorary Member of many other Engineering Institutes and Societies. He presided over the International Electrical Congress at St. Louis in 1904, and the International Electro-technical Commission, 1908–1911, besides taking a prominent part in many other technical conferences both in America and in Europe. He was awarded the Grand Prix in Paris (1889 and 1900) for his electrical inventions, and has received amongst other medals the Rumford Medal (1901), the first Edison Medal (1909), the John Fritz Medal (1916), the Hughes Medal of the Royal Society (1916), the Kelvin Medal (1923), the Faraday Medal (1927), and the Grashof Medal of the Society of German Engineers (1935). In 1924 he received the degree of Doctor of Science from the University of Manchester. Numerous honours were also conferred upon him in America.

He married in 1884 Mary Peck, of New Britain, Conn., by whom he had four sons. She died in 1916, and in 1923 he married Clarissa Hovey, of Boston, Mass., U.S.A.

MADE AND PRINTED IN GREAT BRITAIN BY WILLIAM CLOWES AND SONS, LIMITED, LONDON AND BECCLES.